Assessment of Geotechnical related Problems in Water Retaining Tanks Constructed on Expansive soils in KWWTP

COLLEGE OF ARCHITECTURE AND CIVIL ENGINEERING

Assessment of Geotechnical related Problems in Water Retaining Tanks Constructed on Expansive soils Case study in Kality Waste Water Treatment Plant Construction Project

An Independent Project Submitted to

the school of Graduate Studies of Addis Ababa Science and Technology University in Partial Fulfillment of the Requirements for the Degree of Master of Engineering in college of Architecture and Civil Engineering

(Geotechnical Engineering)

By Abraham Kedru

Sept, 2017

DECLARATION

I, the undersigned, declare that this project is my original work performed and has not been presented as a project for a degree in any other university. All sources of materials used for this thesis have also been duly acknowledged.

Abraham Kedru

Signature: ___________

Place: Addis Ababa Science and University (AASTU)

Addis Ababa.

Date: September/2017

Acknowledgement

I would like to extend my sincere thanks to Dr. Melaku for helping me accomplish this work. I would also like to thank Dr. Fitsum for his invaluable support and advices during my study showing me the way to accomplish it in the best possible way I could; and Addis Ababa Science and technology University school of Civil Engineering and construction technology management, Geotechnical engineering staff members, who gave me comments and directions on my study title using their long experience and deep knowledge on the field of Geo-technical Engineering.

Finally I would like to express my sincere thanks to my family and friends for their encouragement during my study.

ABSTRACT

Damage due to soil swelling and shrinkage is very noticeable in wide spectrum of structures such as roads, buildings, canal linings, reservoirs, landfill liners, etc. So many damages have been reported as a result of expansive soil. Such damages occur when the pressure exerted by the soil is greater than the foundation pressure. The main reason of swelling behavior is water absorption of soil mass in time. And the time required for completion of swelling is relatively long. The soil will also shrink when it loses its moisture which leads to settlement. In this document, it has been tried to assess the extent of the likely damages caused by expansive soil on waste water retaining tanks constructed in KWWTP and conclusion is made that the main cause of failure is fluctuation of ground water table in the black cotton soil on which the tanks are constructed. Finally based on the causes of failures and other factors attempt was made to propose preventive measures for the tanks constructed in expansive soil locations of the study area.

Table of Contents

List of Figures

Acronyms

USCS Unified Soil Classification System AASHTO American Association of State Highway and Transportation Officials

Hd Depth of desiccation

Hs Depth of seasonal moisture fluctuation

Qu Ultimate bearing capacity

Qs Ultimate skin friction

Qb Ultimate base resistance

Cu Undrained shear strength of the soil along the pile shaft α Adhesion factor

Ab Pile base area

Nc End bearing capacity factor

Cub Undrained shear strength of the soil below the pile base

Fu Soil uplift pressure

β Uplift factor

SP Soil swelling pressure

Za Depth of active zone

Qd Super structure load
ω Natural moisture content

LL Liquid limit

PI Plasticity Index

NMC Natural Moisture Content

CL Inorganic clay of low plasticity

OL Organic silt or clay of low plasticity

CH Inorganic clay of high plasticity

OH Organic silt or clay of high plasticity

MH Inorganic silt of high plasticity

KWWTP Kality Waste Water Treatment Plant

Chapter 1: Introduction

1.1 Background

Expansive soils are clay soils that contain clay mineral called Montmorillonite. Such soils are capable of absorbing great amount of water and expand. The expansive nature of the clay is less near the ground surface where the profile is subjected to seasonal and environment changes. The more water they absorb the more their volume increases. Expansive soils also shrink when they dry out. Fissures in the soil can also develop. These fissures help water to penetrate to deeper layers. This produces a cycle of shrinkage and swelling that causes the soil to undergo great amount of volume changes. This movement in the soil results in structural damages especially in lightweight structures such as one or two story buildings, warehouses, retaining walls, sidewalks, driveways, basement floors, pipelines and foundations.

Engineering problems due to expansive soils have been reported in many countries all over the world, costing millions of dollars due to severe damages of structures. These damages are most common especially in the arid and semi-arid regions. Damages are usually manifested through crack of floors and walls, stacked windows and doors, bulged floors and tilted walls and structures. The magnitude of the damages can be extended even to the extent of failure of one or all structure by decreasing the structural safety of the building. Maintenance and repair cost can also exceed the original cost of the foundation and creates financial burden to the owner. Generally, the damage will result in economic loss for building owners and the country at large. (TIBEBU, 2015)

Thus, this study has identified likely damages to be caused by expansive soil the hydraulic tanks constructed in KWWTP and will also provide procedures and methodologies to prevent damage to the already constructed structures.

1.2 Objectives of the study

The main objective of this study is to assess the potential future damages of the tanks constructed on expansive soils in KWWTP and propose the possible and feasible preventive measures for the damages not to happen.

To achieve the main objectives, the study will have the following specific objectives.

- Evaluate and predict the nature of damages
- Recommend methods to prevent the damages and remedial measures

1.2 Scope of the study

This study will assess the likely damages and failures in hydraulic tanks constructed in KWWTP.

1.3 Organization of the study

This document is organized in six chapters as follows. Chapter 1 presents general description about the document. This includes background, general & specific objectives and thesis overview.

Chapter 2 presents literature review on properties of expansive soils, their distribution around the world, characteristics of Ethiopian expansive soils, factors affecting swelling behavior of these soils, types of distresses caused by them, possible causes of damage, methods used to prevent damage due to expansive soil and remedial measures.

Chapter 3 discusses the methodologies followed to collect the required data, how the sample size has been determined and the implemented method of analysis.

4 Chapter 4 discusses the data collection, analysis and results. This includes summarizing all previously conducted researches in Ethiopia and Addis Ababa in relation to expansion soil, analyzing the collected sample data, the design (geotechnical parameters to design the tank), the actually occurring condition in the study area

Chapter 5 discusses the whole site which this study focus on and the subsurface conditions under each major tanks of the plant and the defects and the possible causes.

Chapter 6 contains conclusions and recommendations.

The last sections contain references used in this thesis work and annexes.

Chapter 2: Literature Review

2.1 General

Soil is a mixture of various sizes of particles like gravel, sand, silt and clay. Gravel and sand are the coarse fractions and they are considered inert materials because of their insignificant activity. In contrast, clay and silty clays are particles of ultra-fine size in the form of platelets. They carry an unbalanced negative electric charge on their surface. This electric charge and large specific surface they possess render them highly active. They can absorb water as well as the positively-charged ions from the salts in water to neutralize the electric charge they carry on their surface. The amount of water absorbed depends on the type of the clay mineral present in the soil. Three most common minerals present in clay are Kaolionite, Illite and Montmorillonite and their capacity to adsorb water increases in that order, therefore, the greater the percentage of Montmorillonite mineral present, the greater would be the expansive nature of the soil (Venkataramana, 2003)

2.2 What are expansive soils?

Expansive soils are clay soils containing considerable amount of Montmorillonite mineral which has a potential for shrinking or swelling due to changes in its moisture content. Expansive soil can be classified in to two groups with respect to the parent materials. The first group comprises the basic igneous rocks, such as the basalts of Deccan plateau in India, the Dolerite sills and dykes in the central region of South Africa and Gabbros and norities of west of Pretoria north, Transvaal. In these soils the Feldspar and Pyroxene minerals of the parent rocks have decomposed to form Montmorillonite and other secondary materials. The second group comprises the sedimentary rocks that contain Montmorillonite as a constituent which breaks down physically to form expansive soils. In North America, bedrock shale found in the Pierre Formation and more recent Larame and Denver Formations are examples of this type of rock. In Israel, there are the marls and lime stones and in South Africa, the shale of the Ecca Series. (TIBEBU, 2015)

2.3 Why are expansive soils problematic?

In wet seasons, expansive soil will absorb water and swell up; as a result, the whole ground level rises. This increase in ground level is usually called free-field heave. However, if a structure is built on such a soil deposit, the foundations form an obstruction to the soil to freely move up and consequently, the soil applies an upward pressure on the foundation. This pressure that the soil applies on the foundation is called swell-pressure. If the footing transfers a downward stress which is smaller than the swelling pressure, the footing moves upward. These upward and downward movements of foundations become cyclic seasonal movements during the entire life span of the structure. These cyclic movements tend to tear up the walls and eventually destabilize the whole structure. Light structures, such as single or double storied residential buildings, pavements, etc. which generally transmit smaller stresses to the soil than the swell-pressure are those that suffer the damage most. Once the structure develop cracks, it is hardly possible to rehabilitate it without significant expense. Building a house for most people is a lifetime venture and if it occurs on an expansive soil, the investment needs to be safeguarded. Therefore, there is need for the public to be aware of the implications of building on expansive soils, in order to identify the problem at an early stage rather than regret later. (Venkataramana, 2003)

2.4 Distribution of expansive soils around the world

Expansive soils are found throughout many regions of the world, particularly in arid and semiarid regions, as well as where wet conditions occur after prolonged periods of drought. Their distribution is dependent on geology (parent material), climate, hydrology, geomorphology and vegetation. As shown in figure 2.1 expansive soils are widespread in the African continent, occurring in South Africa, Ethiopia, Kenya, Mozambique, Morocco, Ghana, Nigeria etc. In other part of the world cases of expansive soils have been widely reported in USA, Australia, Canada, India, Spain, Israel, Turkey, Argentina, Venezuela etc. (Leikun, 1999). In these countries, or significant areas of them, the evaporation rate is higher than the annual rainfall creating a moisture deficiency in the soil. Subsequently, when it rains the ground swells and increases the potential for heave to occur. In semi-arid regions a pattern of short periods of rainfall followed by long dry periods (drought) can develop, resulting in seasonal cycles of swelling and shrinkage.

Figure 2.1 Distribution of expansive soils in Africa (Leikun, 1999)

2.5 Distribution of expansive soil1s in Ethiopia

Expansive soil is known to be widely spread in Ethiopia. Although the extent and range of distribution of this problematic soil has not been studied thoroughly: the southern, south-east and south-west part of the city of Addis Ababa areas and central part of Ethiopia following the major trunk roads like Addis-Ambo, Addis-Woliso, Addis-Debre Berhan, Addis-Gohatsion, Addis- Modjo are some of the areas covered by expansive soils. Areas like some part of Mekele, Gondor, Bahirdar, Debreberihan and Gambela are also known to be partly covered by expansive soils (Zewdie.A., 2004)

2.6 Soil-moisture relationship of expansive soil

2.6.1 Effect of stratum thickness

Laboratory research has been done by Chen (1975) to explore the effect of stratum thickness on the amount of volume change and swelling pressure. In this series of tests, the sample thickness ranged from ½ to 1½ inches. The samples were compacted to uniform moisture content & density and sufficient time was allowed for complete saturation of the thickest sample. The test result showed that the magnitude of the volume change is proportional to the sample thickness and the percentage of volume increase remains constant. (TIBEBU, 2015)

2.6.2 Effect of moisture content and dry density

Expansive soils will not be subjected to volume change unless there is an increase in moisture content. A drier soil will swell more than a wet soil. Chen has further extended his laboratory research to determine the effect of increasing the initial moisture content on the volume change as well as swelling pressure. Different samples were compacted at constant density but varying moisture contents and the following results were observed:

- Soil with low moisture content swells most

- The swelling pressure required for zero volume change remained practically constant

- For moisture content slightly higher than optimum moisture content, the volume change should be negligible

As indicated above high moisture content soil will experience less uplift, but the pressure required to maintain constant volume change will not be altered. This also indicates that the commonly accepted procedure of prewetting the foundation excavation to eliminate the swelling characteristics is not a reliable procedure. Prewetting can only serve to decrease the amount of swelling. A foundation placed on such soil will still require the same amount of dead load pressure (TIBEBU, 2015)

Researches also show that when dry density decrease, swelling pressure rapidly approaches zero and when dry density increases, swelling pressure rapidly increases and approaches infinity. Therefore, swelling pressure of clay soil is independent of initial moisture content, degree of saturation and thickness of the stratum. Therefore, dry density is the only factor that affects swelling pressure of clay soils. (TIBEBU, 2015)

2.6.3 Effect of depth of moisture fluctuation

The thickness of expansive soil layer below the ground surface and the depth to which the season moisture varies, called the ‘Active depth’, greatly influence the amount of heave the ground would undergo. Figure 2.2 shows the seasonal moisture variation and the active depth of homogeneous soil. Below the active depth, the soil water content is considered to remain constant; therefore, foundations situated within the active depth are subject to distress from soil volume change (Venkataramana, 2003)

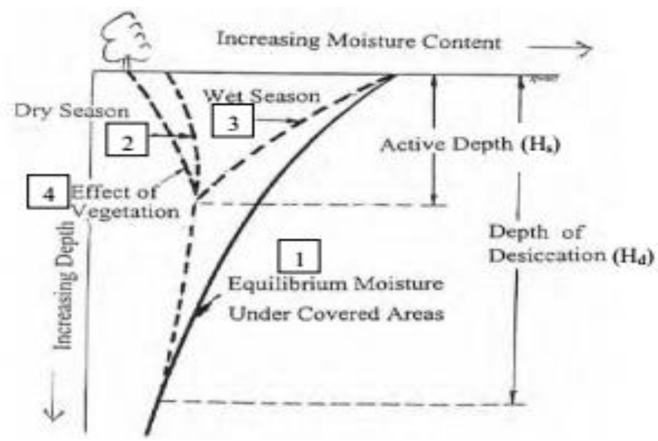


Figure 2.2 Seasonal moisture variations (Venkataramana, 2003)

In a covered area, the moisture profile is shown by curve 1. There is no gain or loss of moisture to the atmosphere. The moisture content of the soil deceases with depth. Curve 2 indicates the moisture content variation in uncovered natural condition. Evaporation causes loss of moisture in the soil near the ground surface. However, the influence of evaporation decreases with depth and at some depth, Hd, the moisture content equilibrium remains the same as the covered. The value of Hd depends on the climate condition, the type of soil and the location of the water table. This depth represents the total thickness of material which has a potential to expand because of water deficiency. It is impossible to determine the value of Hd. the hotter and drier the climate, the greater the depth of desiccation. The maximum depth of Hd is equal to the depth of the water table and the minimum depth is equal to the seasonal moisture fluctuation depth described below. (TIBEBU, 2015)

During wet months with heaver precipitation and higher humidity, the moisture content of near surface soil increases and the moisture profile represented by curve 2 alters its shape to curve 3. Another case is plantation of trees and shrubs which further reduce the moisture content of the soil and this condition is designated by curve 4.

The depth of seasonal moisture content fluctuation, Hs, indicated in figure 2.2 depends on the variation of surface moisture, permeability of the soil, and climatic conditions. In areas where precipitation and evaporation are fairly constant, the Hs depth may be only few meters. When a long drought is followed by an intense rainfall, the Hs depth will be very deep.

It should be noted that man-made environments such as watering of lawns, discharge of roof drains, formation of drainage channels and swales, and possibility of utility line leakage will also increase the value of Hs. (TIBEBU, 2015)

2.7 Types of foundation distresses

When a building is constructed on an expansive ground surface evaporation and temperature variations are retarded below it. As a result, patterns of soil movement under the building are identified on the basis of short-term and long-term effects. This soil movement will create the following foundation distresses:

2.7.1 Edge heave

In the short-term, i.e., just after the construction is complete, below the center of the building, moisture variation remains small while at the edges, seasonal moisture change continue to occur. If the building is constructed during the dry season, in the wet season that follows, soil around the building absorbs rain water and swells pushing up the peripheral foundations. This effect is called the edge heave (Tefera.A, 2008). Edge heave can also be initiated due to local effects, such as, sprinkling of water for vegetation around building, leaking utility lines and ponding of surface water due to poor drainage system.

In addition to moisture variation, edge heave can be initiated by effect of confinement of the foundation system. For example, the exterior corners of a uniformly-loaded rectangular slab foundation will only exert about one-fourth of the normal pressure on a swelling soil of that exerted at the central portion of the slab. As a result, the corners tend to be lifted up relative to the central portion (Rogers.J.D)

In this case, the cracks develop from the lower region and transverse diagonally upward being wider at the bottom and narrower at top.

2.7.2 Edge shrinkage

Like edge heave, edge shrinkage is also a short-term effect. If the building is constructed during the wet season, in the dry season that follows, soil around the building loses the absorbed rain water and the soil leaves the peripheral foundations. This effect is called the edge shrinkage. Presence of big trees closed to building will also cause edge shrinkage. Especially during the dry season when moisture available for roots to suck is the least, trees absorb water from the nearby foundation soil through their root system and cause shrinkage of soil.

In the edge shrinkage condition, cracks propagate diagonally across from top to bottom being wider at the top and narrower towards the bottom of the structure (Venkataramana, 2003).

2.7.3 Central heave

In the long-term, moisture starts continuously accumulating under the center of the structure being drawn up from the ground water at depth by capillary action until an equilibrium moisture condition is attained. This means that the soil under the building continuously swells with time and finally attains a mound shape while at the edges; the seasonal effects continue to occur. This is termed the central-heave condition. As a consequence of this, the foundation gets only a partial support from the soil at the center as shown. The height of the mound formation depends on the area of the soil covered by the building and also the active depth. Cracking of walls start in the upper region and traverse downward direction.

2.8 Identifications of damage due to expansive soil

The most obvious identifications of damage to buildings are doors & windows that get jammed, uneven floors and cracked foundations, floors, masonry walls, ceilings & pavement around buildings (Lucian.C, 2006).

The degree of damage based on observed cracks ranges from hairline cracks, severe cracks, very severe cracks to total collapse. As it has been explained in previous section the pattern of the cracks depends on whether it is a dooming heave or a dish shaped lift heave. In both cases vertical, horizontal and/or diagonal cracks will be developed along walls and floor area. This intern has a great effect on functionality of doors and windows of buildings.

2.9 Possible cause of damage

2.9.1 Climate

Seasonal changes in rainfall were typically the principal cause of the change of soil moisture. This led to downward movement during summer and upward movement during winter. The consequent rising and settling of ground surface occurred in the dry and wet seasons resulting in seasonal subsidence and seasonal recovery respectively.

The results and observations by most of the researchers suggested that expansive soils which experienced periodic swelling and shrinkage during alternate wet and dry seasons caused considerable damage to structures founded on them. The damage to structures built on expansive soil in wet climates usually occurred during drought period and damage to structure built on dry climate occurred during rainy season (Osman, 2005).

2.9.2 Poor drainage system

Improper drainage is probably the most important factor contributing to soil volume change and subsequent damage to pavements. If water is allowed to stand in drainage ditches close to buildings, it can penetrate down and amplify heave (Nelson, 1991).

The following can be considered as the main causes of poor surface drainage:

- Surface runoff not properly drained away from the building
- Sprinkling of water for grass and shrub plantation
- Overflow from elevated and/or ground water tank
- Slope of surrounding area (outside the compound)

2.9.3 Presence of vegetation and big trees

Existence of vegetation, such as fast growing trees in the vicinity of compound walls can sometimes cause cracks in walls due to expansive action of roots growing under the foundation. Roots of a tree generally spread horizontally on all sides to the extent of height of the tree above the ground and when trees are located close to a wall; these should always be viewed with suspicion (Leikun, 1999).

Trees absorb water from the nearby foundation soil through their root system and cause shrinkage of soil especially during the dry season when moisture available for roots to suck is the least. This is the reason why big trees should not be located within a distance of 1 to ½ times their mature height from the structure, if big trees are felled just before construction, transpiration loss of moisture through leaves is discontinued and the soil moisture accumulates and allows the soil to swell. Therefore, trees have to be cut down far in advance of construction so that soil moisture condition reaches an equilibrium condition. If trees are retained, moisture barriers should be put in place (Venkataramana, 2003).

2.9.4 Damaged utility structures

Shallow plumbing pipes buried in the zone of seasonal moisture fluctuation are exposed to enormous stresses by shrinking soils. If water or sewage pipes break, then the resultant leaking moisture can aggravate swelling damage to nearby structures (Lulu, 2010).

The effect of a leaking water line is dependent on the soil moisture condition in the supporting expansive soil mass prior to the leaking occurrence. Dishing of floor systems due to clay heave under the footings could occur when excessive water is present due to site leakage at the edges of the structure (Osman, 2005).

2.9.5 New construction near existing building

When areas are covered by new structures such as buildings, pavements, sidewalks or aprons, evaporation is blocked or is partially retarded. The moisture content beneath the covered area increases due to gravitational migration, capillary action, vapor and liquid thermal transfer and, in the course of several years, the depth of seasonal moisture content fluctuation Hs can approach to depth of desiccation Hd (Tessema.G., 1984).

In the same manner if structures around the building have been demolished due to various reasons then covered area around the building will be reduced. This leads to migration of moisture away from building foundation soil until new soil moisture equilibrium has been reached.

Such conditions cause foundation distress depending on type of foundation, moisture content and swelling potential of the soil.

2.9.6 Creep action – foundation on steep slope surface

Expansive soils on slope tend to move down the slope due to soil creep especially when the soil is wet. Soil creep is a very slow movement down the slope, which is not perceptible. However, overtime it can be significant and can damage the foundations. The steeper the slope, the greater is this effect. Furthermore, on slope, these soils can cause landside problems. Slopes steeper than about 14o (a residual friction angle of soil) have the tendency to slide, if conditions are favorable (Venkataramana, 2003).

2.9.7 Construction interruption

Due to financial problem of homeowners it is a common trend to construct foundation or substructure part in one phase and super structure in the next phase. Buildings are designed to the final stage or condition. Because of construction interruption, the dead load pressure which counterbalances the uplift force gets lower resulting in foundation movement.

This results a failure even if the foundation is designed properly with the precautions to be taken for expansive soil (Sissay, 2008).

2.10 Method of preventing structure damages

In order to minimize or eliminate the danger of damage of buildings because of heave and shrinkage, the following methods shall be used:

2.10.1 Moisture control

The main cause of heaving and shrinkage is the fluctuation of moisture under and around the structure in question. In any site, depending up on the topography, geological and weathering conditions, the natural ground water fluctuates. In a country like Ethiopia, where there are distinct dry and wet seasons, the fluctuation of ground water table during these periods is large. In addition to the fluctuation of the ground water one should also consider free water which may seep under foundations, or the effect of evaporation which would cause moisture migration. (TIBEBU, 2015)

Hence a satisfactory solution to the problem would be to device an economical way of stabilizing the soil moisture under and around buildings. It doesn't matter whether the moisture content is high or low, as long as it can be maintained constant throughout the year. This moisture fluctuation can be controlled by using the following methods:

Horizontal moisture barriers

Horizontal moisture barriers can be installed around a building in the form of membranes, rigid paving, or flexible paving. The purpose of the horizontal barriers is to prevent excessive intake of surface moisture (TIBEBU, 2015). Widely used horizontal membranes are polyethylene membrane, concrete aprons and asphalt membrane, extending beyond the limits of backfill (Tefera, 2008).

Vertical moisture barriers

Vertical moisture barriers function in much the same way as horizontal barriers in terms of slowing the rate of heave and causing the water content distribution to be more uniform below the structure. However, vertical barriers are more effective than horizontal barriers in retarding lateral moisture migration. Consequently, edge effects are minimized (Nelson, 1991). To severe as a barrier in this category, one may use polyethylene membrane, concrete, or other durable impervious material. Depth of the barriers should be equal to or greater than the depth of moisture fluctuation.

Backfill materials may be used as vertical moisture barriers provided they are well compacted (Tefera, 2008).

Subsurface drainage

The purposes of a subsurface drainage system are as follows:

1. Intercept the gravity flow of free water,

2. Lower the ground water or perched water, and

3. Arrest the capillary moisture movement and movement of moisture in the vapor state.The following are the major sub surface damage system.

a) Intercepting drains; Intercepting drains are effective in minimizing the wetting of the foundation soils where the wetting is due to the gravity flow of free water in a subsurface pervious layer such as a layer of gravel or fissured clay. To insure the interception of free water, the drain must be completely filled with gravel and trench should be deep enough to reach the water-bearing layer.

b) Peripheral drains; Peripheral drains can be installed around either the interior or exterior of the building. The sub drainage system is effective in minimizing general wetting of foundation soils, which occur not only because of gravitational flow of free water, but also because of moisture migration.

c) Surface drainage; The ground surface around a building should be graded so that surface water will drain away from the surface in all directions. This usually is not accomplished due to negligence, cost, limited property size and other reasons. As a result, it is not uncommon to find buildings with surface drainage directed towards the foundation walls. Moisture change at the perimeter of the building appears to be the most significant contributor to damage. Therefore, by improving the drainage, a beneficial effect is inevitable (TIBEBU, 2015).

2.10.2 Soil stabilization

The successful application of soil stabilization procedures require considerable experience and judgment regarding the soil on-site, consideration of limitations of the methods to be chosen, and correct implementation procedures. Treatment procedures that are available for stabilizing expansive soils before and after construction of structures and highways include the following:

Soil replacement

Soil replacement is the simplest and easiest solutions for slabs and footings founded on expansive soil. The expansive foundation soils are replaced by non-heaving materials. The strength of the method lies in the selection of the replacement material and depth of replacement.

If the active zone is very deep, it is not desirable that moisture migrate to the underlying expansive soil. For such condition material selected for replacement should be impervious non-expansive soil. Engineering judgment should be used in deciding on the thickness of the replacement.

It should always be remembered that the replaced soil serves as a cushion, and even if the deep-seated soil swells, the movement due to heave will be regulated (Tefera.A, 2008).

Pre-wetting or flooding

Pre-wetting is an old established concept among engineers and contractors as well as laymen in dealing with swelling soils. Pre-wetting or ponding/flooding/ is based on the theory that increasing the moisture content in the expansive foundation soil will cause heave to occur prior to construction and thereby eliminate problems afterward. It is assumed that if the high moisture content is maintained, there will be no appreciable increase in soil volume to damage the structure (Nelson, 1991).

This procedure may have serious drawbacks that limit its application. Expansive soils typically exhibit low hydraulic conductivity and the time required for adequate wetting can be up to several years. Furthermore, after the water has been applied for long period of time serious loose of soil strength can result causing reduction in bearing capacity and slope instability. Another major drawback to the use of this procedure is that after a prolonged period of surface ponding the wetting front of the infiltrating water will have advanced to only to a depth much less than of the active zone. Redistribution of water throughout the active zone can continue after construction due to high water content in the zone above the wetting front. The continued migration of water into lower layer can result in continued heave after construction (Nelson, 1991).

Pre-wetting shall be used for stabilizing soil beneath floor slabs, pavement or canal linings. However, its application for building foundations is still questionable and risky (Tefera.A, 2008).

Chemical stabilization

By chemical stabilization is meant the process of mixing additives like lime, cement, organic and inorganic chemicals to expansive soils, so as to retard their potential expansiveness (Tefera, 2008).

Lime stabilization has been used successfully on many projects to minimize swelling and improve soil plasticity and workability. The addition of lime reduces the plasticity of the soil and hence it's swelling potential. Generally, from 3 to 8% by weight hydrated lime is added to the top of several centimeters of the soil. The strength characteristics of a lime-treated soil depend primarily on soil type, lime type. Lime percentage, and curing conditions (time temperature) (Nelson, 1991).

Some organic and inorganic chemicals have also been tested for their effectiveness in stabilizing expansive soils. Most of the chemicals were tested in laboratories. Their economic use in the field has not yet been reported (Tefera, 2008).

2.10.3 Compaction control

The amount of swelling that occurs when a structural fill is exposed to additional moisture depends majorly upon the compacted dry density and the moisture content.

Gizienski and Lee show that expansive clays expand very little when compacted at low densities and high moisture but expand greatly when compacted at high densities and low moisture. For instance, researchers shows that, by decreasing the dry density of typical expansive clay soil from 17.17kn/m3 to 15.7kn/m3, the swelling pressure decreased from 622.6Kpa to 239.5Kpa and the swelling potential decreases from 6.7 to 4.2 percent. All of this can be accomplished without changing the moisture content. The main advantage of using this approach is that the swelling potential can be reduced without the adverse effects caused by introducing excessive moisture into the soil.

The shortcoming of pre-wetting methods mentioned in the preceding section can be eliminated by compaction control. Excess water will not be present in the soil; therefore, there will not be migration of moisture to the underlying moisture-deficient soils and long waiting periods, prior to construction, will be unnecessary. A reasonably good bearing capacity can be assigned to the low density soil (TIBEBU, 2015).

2.10.4 Structural measures

The structural measures that should be undertaken in order to minimize or, if possible, to eliminate damages of structures due to heaving are dependent on the design of the structures. The types of foundations commonly used worldwide to support structural loads in expansive soil environment are: shallow individual or continuous footings, rigid or stiffened raft and bored concrete piles.

The performance and selection of a technically viable foundation type for a certain structure founded on expansive soil will depend on:

- The swelling characteristics of the encountered soils; i.e., the swelling potential of the expansive clay layer(s), depth and layering sequence of the encountered deposits within the influence range of the foundation system.
- The environmental conditions; these include the moisture content of the swelling soil layers, depth of water table, rainfall intensity, temperature and vegetation cover. These factors affect and control the depth of the active clay zone.
- The type of the structure, its shape, rigidity/flexibility and tolerance to movements

- Constructional considerations such as availability of certain construction tools (e.g. piling rigs) and the experience of local contractors and home owners.

Some of the structural measures are discussed below:

Shallow individual or continuous footings

Shallow spread footings typically are not used in expansive soil applications. Where shallow footings are used techniques usually are applied in an attempt to increase the bearing pressure so as to minimize heave.

Some modifications that have been used include (Osman, 2005):

- Narrowing the width of the footing base
- Placing the foundation wall directly on grade without a footing
- providing void spaces within the supporting beam or wall to concentrate loads at isolated points
- increasing the reinforcement around the perimeter and into the floor slab to stiffen the foundation

Shallow footings are preferred where the expansive soil stratum is relatively thin to allow placing the footing on a low expansive or low swelling stratum.

Strip footings are used for load bearing structures but lack the three dimensional rigidity needed to resist small movements. Isolated or pad footings offer some structural rigidity needed to resist small movements therefore perform better than strip footings (Ahmed.M.E., 20011)

Rigid or stiffened mat foundation

Stiffened raft foundation consists of thin concrete slab stiffened with cross beams to provide additional stiffness of the slab. They are applicable with good performance in areas where soils possess large amounts of movements (Ahmed.M.E., 20011).

Pile Foundation

Piles are favored in expansive soils mainly because of their ability to resist uplift forces when properly installed. Piles in expansive soils are designed to act as anchors against uplift forces generated by these soils. They should develop sufficient capacity to carry structural loads and the movement of piles due to the net effect of uplift forces and structural loads should be less than a prescribed limit. The ultimate bearing capacity Qu is the summation of the ultimate skin friction Qs and ultimate base resistance Qb. For a circular pile Qu is obtained using the following equation:

$Q_u = Q_s + Q_b$ . . . . (1)

$Qs = \pi \, d \, L \, \alpha \, Cu$. . . . . (2)

Where:

d: is the pile diameter

Cu: is the undrained shear strength of the soil along the pile shaft

L: is the pile length, and

α: is the adhesion factor

The undrained strength of the soil around the pile shaft is often obtained through laboratory testing of representative soil samples around the pile, whereas the adhesion factor varies according to the soil type, pile type and method of pile installation. The ultimate base resistance is obtained using the following equation:

$Q_b = A_b \, N_c \, C_{ub}$. . . . . (3)

Where:

$A_b =$ is the pile base area

N_c: = is end bearing capacity factor, and

C_{ub}. = is the undrained shear strength of the soil below the pile base

The end bearing capacity factor is a function of the soil type and its friction angle.

The design for uplift generally follows the simplified Chen (1975) method. The uplift forces within the active zone and the withholding (resisting) forces within the anchorage zone. For a safe pile the withholding forces will resist the uplift forces. The method assumes that uplift force is a function of the soil swelling pressure within the active zone. The uplift pressure Fu is the soil swelling pressure (SP) multiplied by an uplift factor (β). The uplift force along the active zone is obtained using the following equation:

F_u. = π d Z_aβ SP. . . . . (4)

Here Z_a is the depth of active zone. Therefore, to compute the uplift force, the designer needs to know the uplift factor, soil swelling pressure within the active zone depth and active zone depth.

The resistance to uplift (W) is offered by the adhesion resistance of the withholding part of the pile (L-Z_a) and by the allowable load from the superstructure (Q_d). It is given by the following equation: W = π d α C_u (L –Z_a) + Q_d. . . . . (5)

A safe design requires that the uplift force (Fu) should be less than or equal to the withholding force or resistance (W). Equations (4) and (5) are equated and solved for the safe pile length. From the above equations, parameters needed for the design of piles in expansive soils are: the adhesion factor α, bearing capacity factor Nc, uplift factor β & active zone depth Za (Ahmed.M.E., 20011).

2.11 Remedial measures

As indicated in previous sections, the building cracks were due to the cycle of heaves and shrink during the wet and dry seasons. The theme of remedial work consist of two major parts: first, to prevent the cycle of heave and shrinkage by protecting the foundation of the building from water content changes; and second, to increase the stiffness of the building by increasing the cross-sectional area of structural elements (such as beams and columns) to account for increased bending moment and shear stress that would result due to the possible differential heave (Bani-Han, 2007) and/or to increase the resistance of structure against the uplift force using different techniques like reducing footing area, providing space under continues footings and so on. Both methods are discussed in detail in the following sections.

2.11.1 Minimizing moisture fluctuation

To keep the moisture content constant by keeping the soil in dry state or saturated state is not practical due to environmental and practical reasons. The problem can, therefore, be best tackled by minimizing the moisture gradient between the edges and the center of the covered area (Sharma, 1987).

To achieve this goal the following measures can be implemented;

- Cut trees and bushes located at a distance ranging from 1.0 to 1.5 times the height of the tree,
- Provide positive drainage around the building,
- Maintain damaged gutters, down pipes, sewerage pipes and water supply pipes that are creating local wetting
- Extend the covered area around the building by providing an apron. An apron at the top of ground is not advisable because it is liable to damage by external forces or differential movement of the ground over which it rests. Therefore providing waterproof apron of about 2 m width at a depth of about 50 cm is more appropriate.

- As explained in detail in section 2.10.1, install horizontal barriers, vertical barriers, intercepting drains, peripheral drains or combination of them depending on source of water and surface drainage conditions (Bani-Han, 2007).

2.11.2 Increasing stiffness of the structure and/or resistance against uplift

In addition to minimizing moisture fluctuation increasing the stiffness of the building and/or resistance against uplift pressure would also be a good solution for buildings damaged by cyclic heave and shrinkage of expansive soils. Such remedial measures various depending on the type of foundation system and extent of damages. Most commonly used structural remedial measures are presented below with respect to foundation systems (Sissay, 2008).

For drilled pier foundation:

- Loosen soil around the pier to reduce the uplift pressure
- Reconstruct void space beneath the foundation slabs
- Eliminate the mushroom at the top of the piers or foundation columns
- Cut the top of the pier and adjust the pier by shims
- Remove all the back fill around the building and replace with compacted non-expansive clay to protect surface water entering through the foundation soil
- Improve the drainage condition around the building by providing adequate slope away from the building and paving with concrete

For continuous footing foundation

- Provide voids beneath the footings at calculated interval to increase the dead load pressure
- Post-tension the foundation to provide structural stability
- Reinforce existing foundation walls with new reinforced grade beam to tie the structure as in box construction
- Under pile the structure with piers drilled in to bed rock

For individual pad foundation:

- Decrease pad size to increase the dead load pressure
- Underpin the pad with piers drilled into bed rock

For basement

- Saw cut the slab along the foundation wall to allow free slab movement
- Adjust screw jack on top of pipe column to re-level interior I-beam
- Provide slip joints to all internal slab bearing partition walls, including door frames and staircase walls

Chapter 3: Methodology and Organization

3.1 Description of the study area.

Kality waste treatment plant is a waste water treatment plant project owned by Addis Ababa City water and sewerage authority. It is a project planned to treat 100,000 m3 of waste water per day. The following are the major treatment units which the plant consists of:

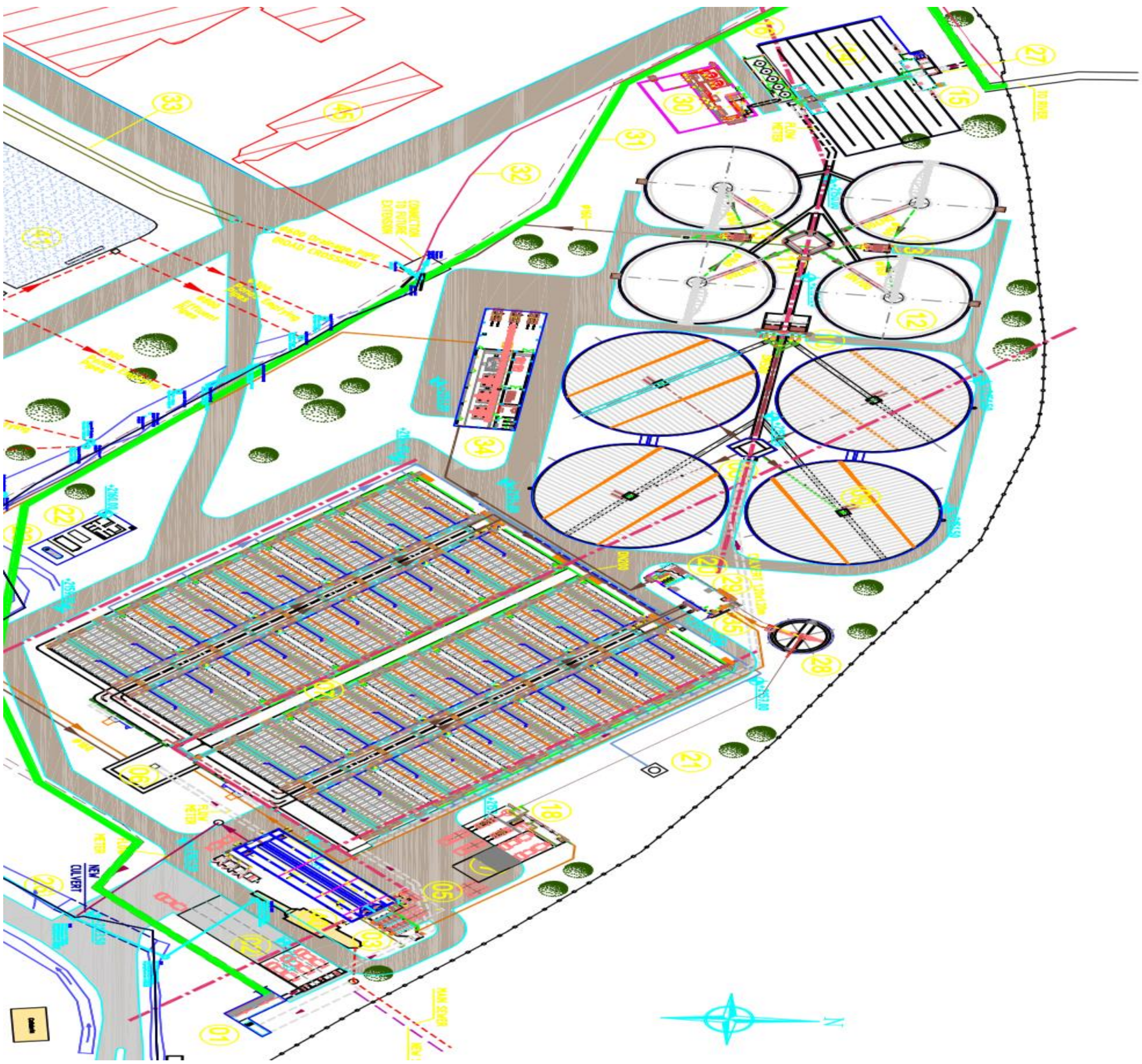


Fig 3.1 General Layout of Kality Waste Waste Water Treatment Plant (AKTOR Technical Societe', 2015)

3.2.1 Inflow Chamber

It is the inlet structure which receives the waste from the sewerage line and directs the flow to the septic waste receiving unit.

3.2.2 Septic waste receiving Unit

This is a tank which receives septic wastes from the upstream inflow chamber and the vacuum trucks and guides the flow to the screening unit.

3.2.3 Screening

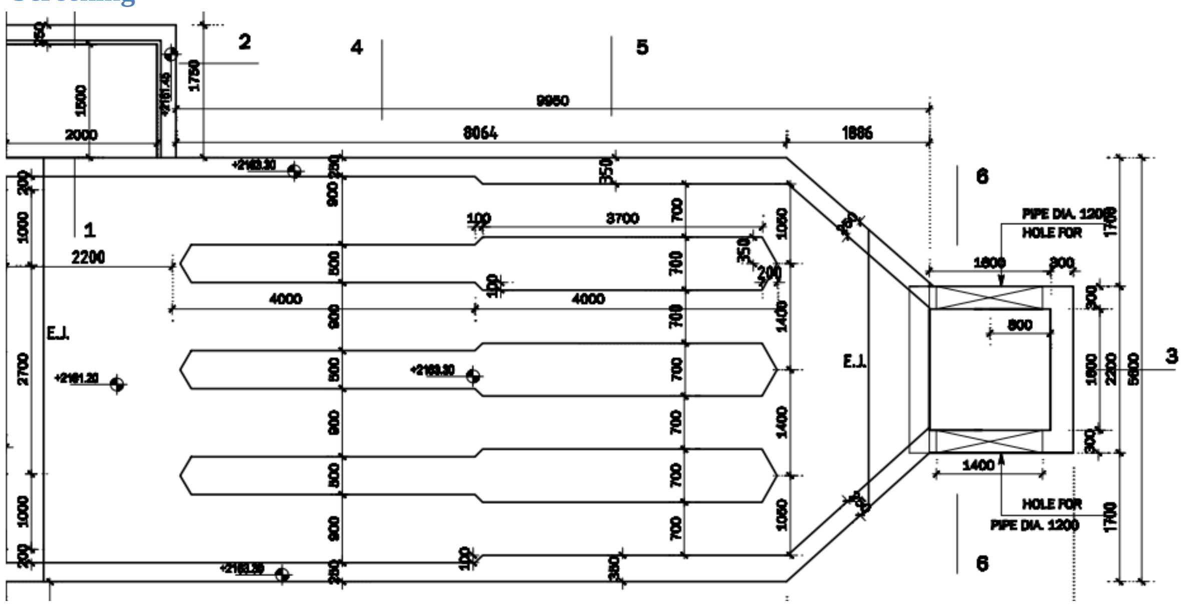


Fig 3.2 Screening unit (AKTOR Technical Societe', 2015)

This is a primary treatment unit where the preliminary treatment process begins. It recives the raw waste water from the sewerage line through the inlet and the septic waste receiving units and screens mostly of big sized particles with the aid of electro- mechanical screening tools guides the flow to the dowenstream unit called Grit and Grease removal Unit.

3.2.4 Grit and Grease removal Unit

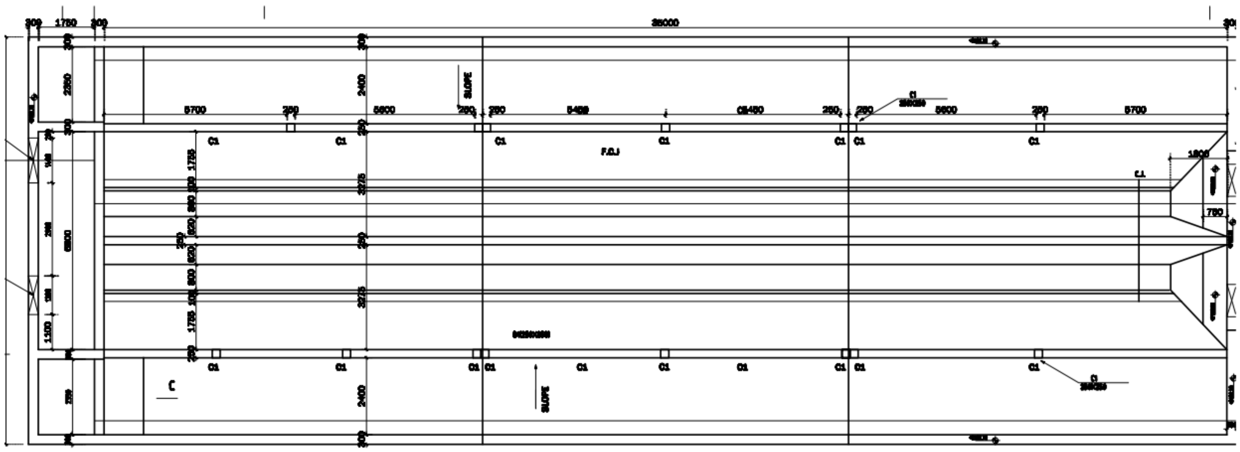


Fig 3.3 Grit and Grease removal Unit (AKTOR Technical Societe', 2015)

This Unit also is a preliminary treatment unit which is mainly designed in order to settle heavy and greased particles through the process of sedimentation. These particles are too small to be screened at the upstream screening station screening tools.

3.2.5 Distribution Chamber

This is a big sized chamber whose main purpose to receive the flow from the upstream units and guide it to the downstream four *UASB Reactors (Up-flow Anaerobic Sludge Blanket Reactors)*. It distributes the flow to the four UASB Reactor units evenly via the distributing channels constructed at the top of the chamber wall.

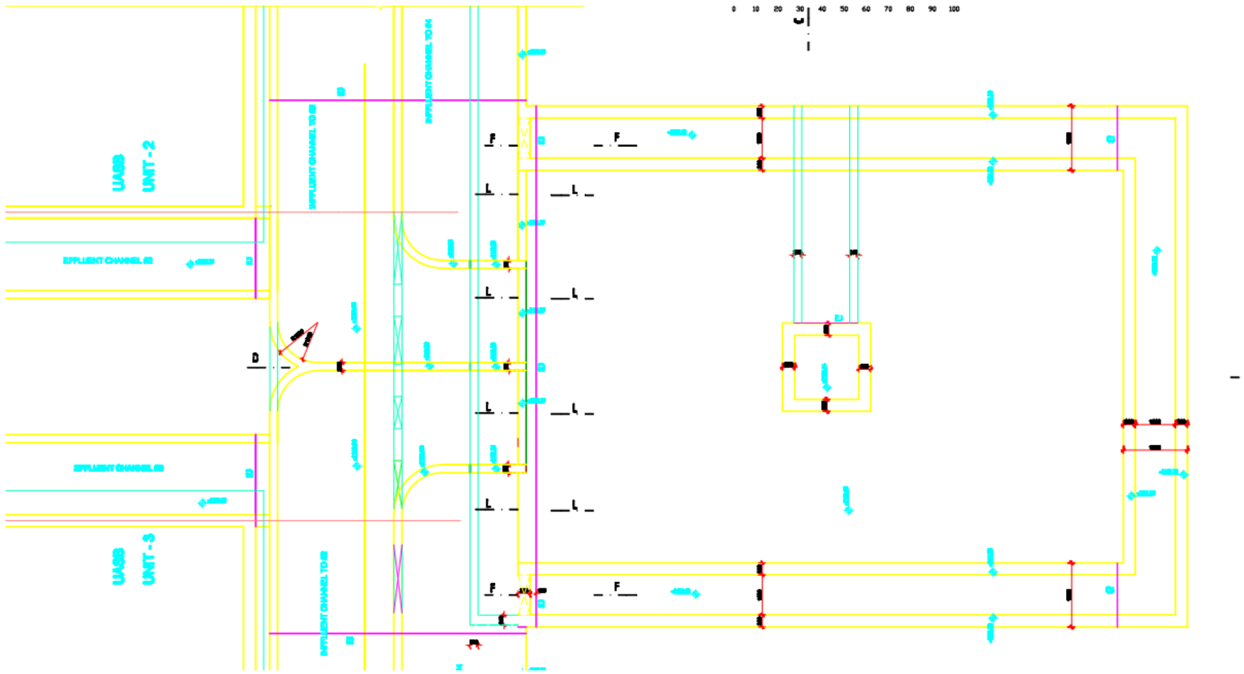


Fig 3.4 Distribution Chamber (AKTOR Technical Societe', 2015)

3.2.6 UASB Reactors (Up-flow Anaerobic Sludge Blanket Reactors)

These are four units of big sizes which are partitioned in to five 20m x 20m x7m sub-tanks each. These units are the main treatment units of the plant. The gourd condition is of unweather basaltic rock nature

such that the condition is of low concern of settlement and bearing capacity problems. However the hydrostatic pressure in these tanks is very heavy so that the foundation is constructed very carefully. The foundation slab thickness is 40cm C-30 concrete with a heavily reinforced section.

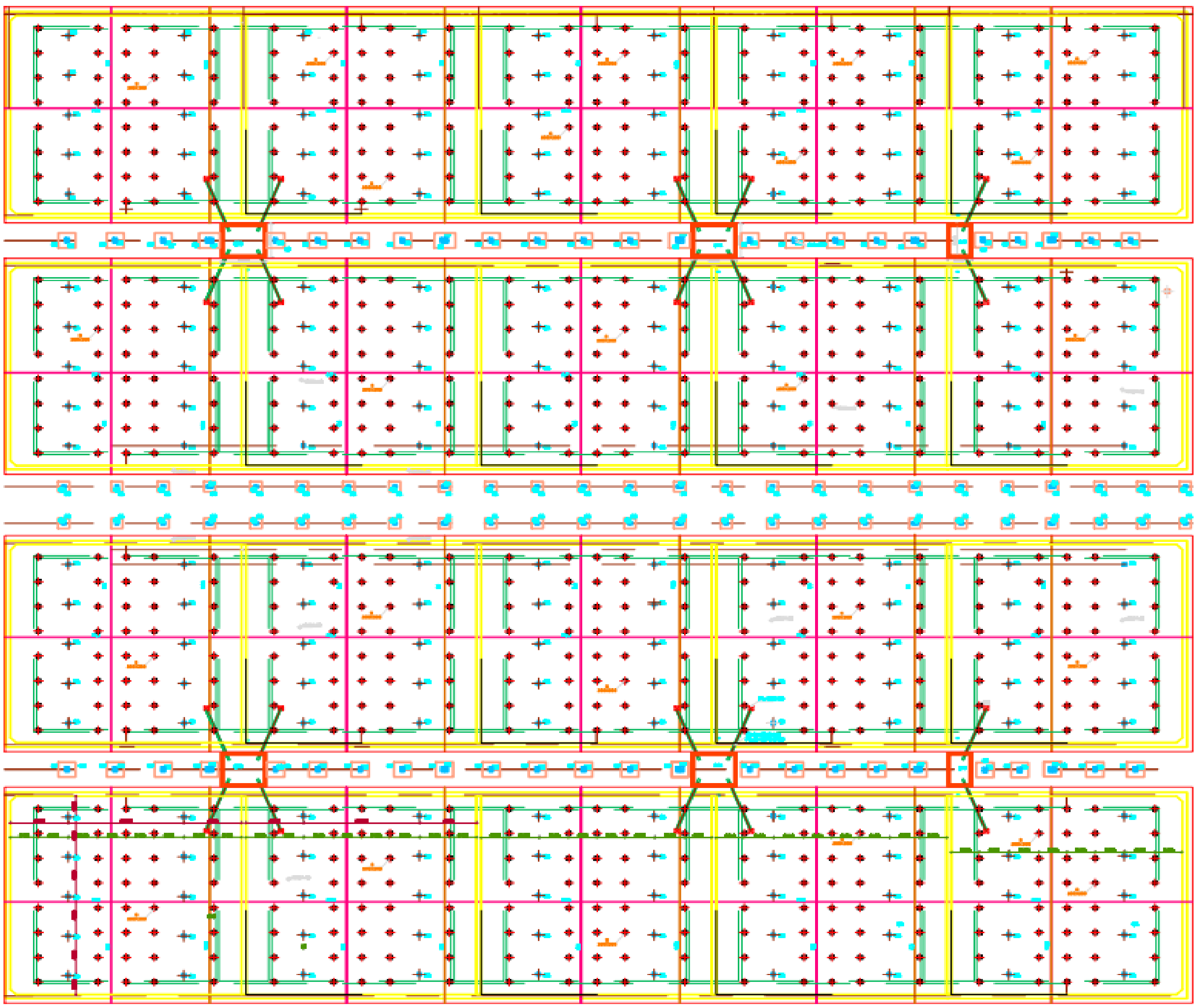


Fig 3.5 UASB Reactors (Up-flow Anaerobic Sludge blanket) (AKTOR Technical Societe', 2015)

3.2.7 Trickling Filters

These also are four circular tanks with an internal diameter of 41m. But these tanks are basically used as filtering tanks of the flow received from the UASB Reactors in which there would not be hydrostatic development so that it can be considered as a showering tank. Therefore the foundation is 30cm C-30 reinforced concrete to make sure waterproofing.

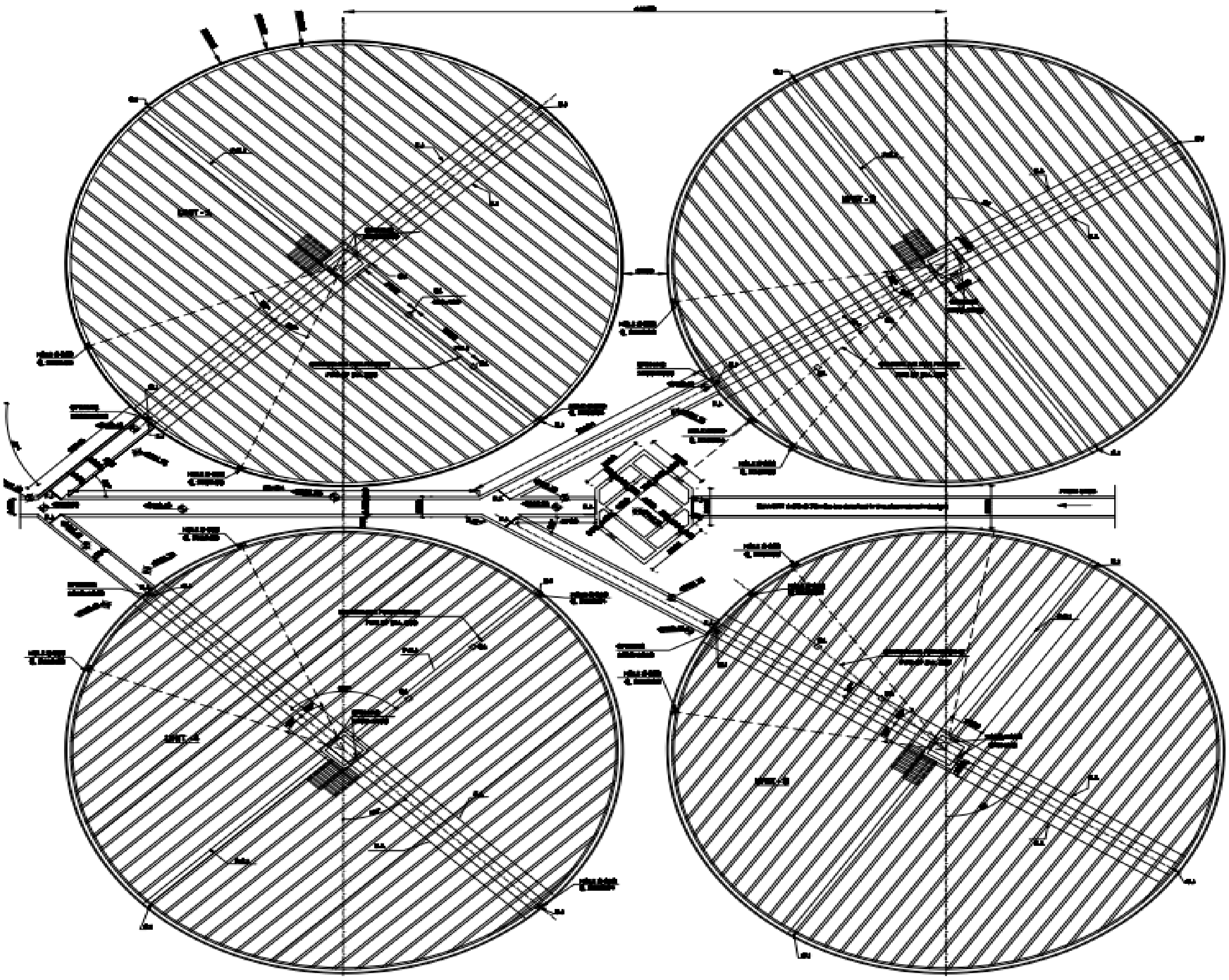


Fig 3.6 Trickling Filters (AKTOR Technical Societe', 2015)

3.2.8 Secondary clarifiers

These are tanks which are designed to further treat the filtered water in the trickling filters by the process of sedimentation. Water in these thanks is fed through the feeder pipes at a central tower and then flow down to the bottom of the tank floor gently so as not to make turbulence in order to make the sedimentation process effective.

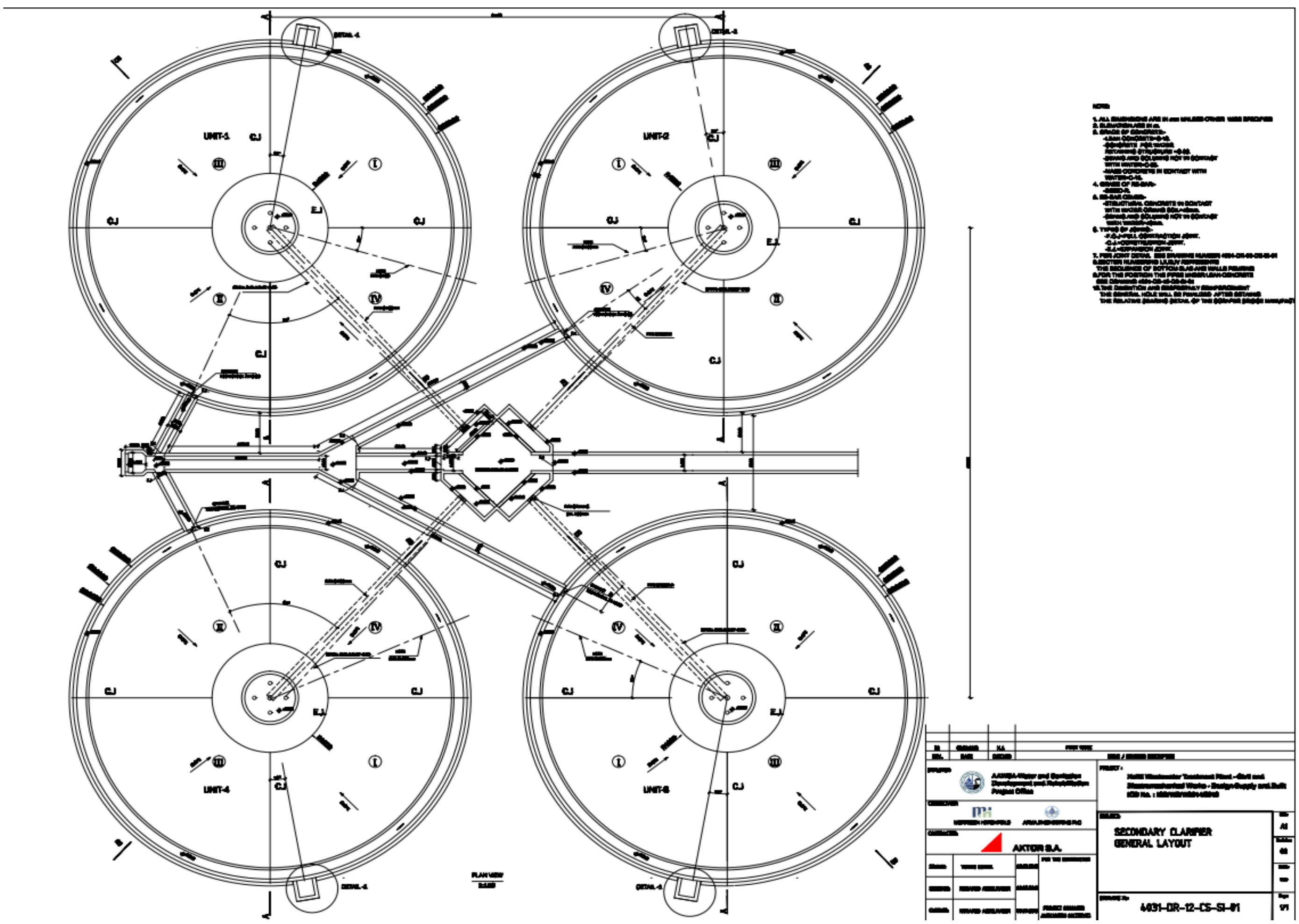


Fig 3.7 General Layout of the four secondary Clarifiers (AKTOR Technical Societe', 2015)

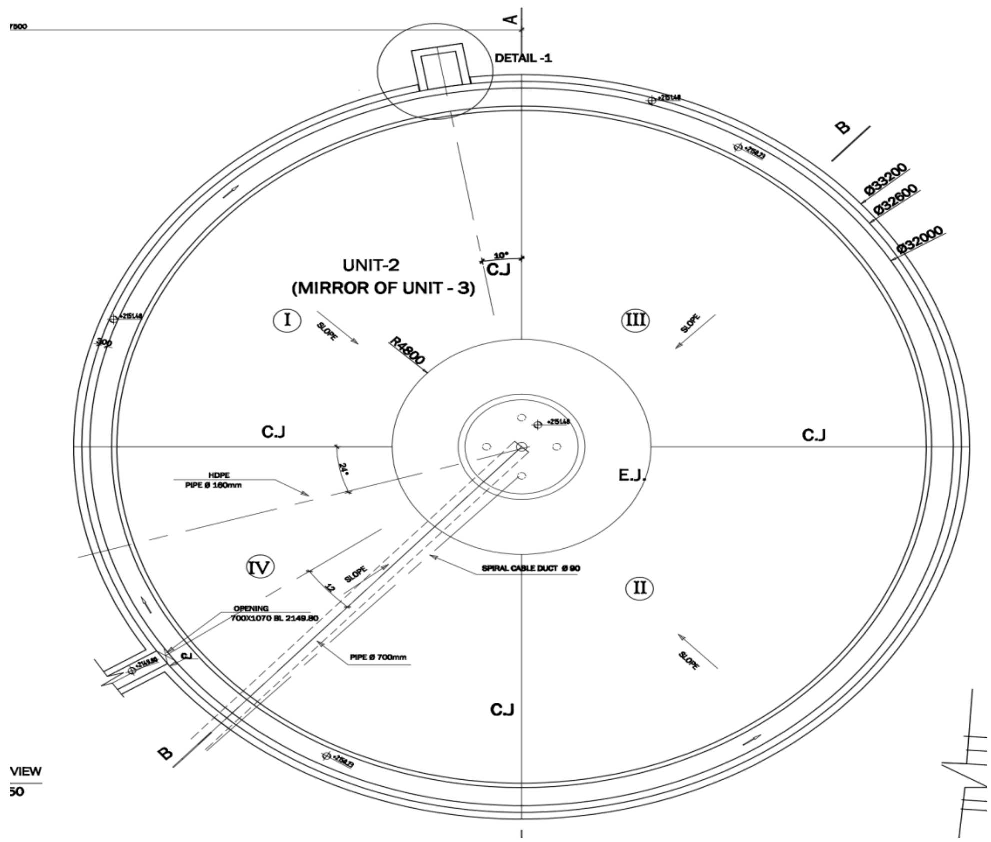


Fig 3.8 Size and dimensions of a single secondary clarifier tank (AKTOR Technical Societe', 2015)

These tanks are the tanks in which this study has a special interest. The main reason behind this special focus is that the soil condition up on which the tanks founded are questionable, which is black cotton soil and it as known is highly expansive in nature.

The construction practice followed to overcome this expansiveness nature is hard to be acceptable due to the fact as shall be shown some laboratory tests the nature of the backfilling material is expansive soil with some granular materials blended.

These tanks consists the 21% of the whole project in size. Functionally they are one of the important units of the plant has. So the construction of these important tanks should have been executed with a special care and attention.

3.2.9 Chlorination

This tank is the last treatment unit in which chemical treatment takes place. The treated industrial water from the upstream secondary clarifier is further treated with the application chlorination and dechlorination process. It is from this tank the treated water is disposed to the nearby reviver via the outlet pipes and chambers.

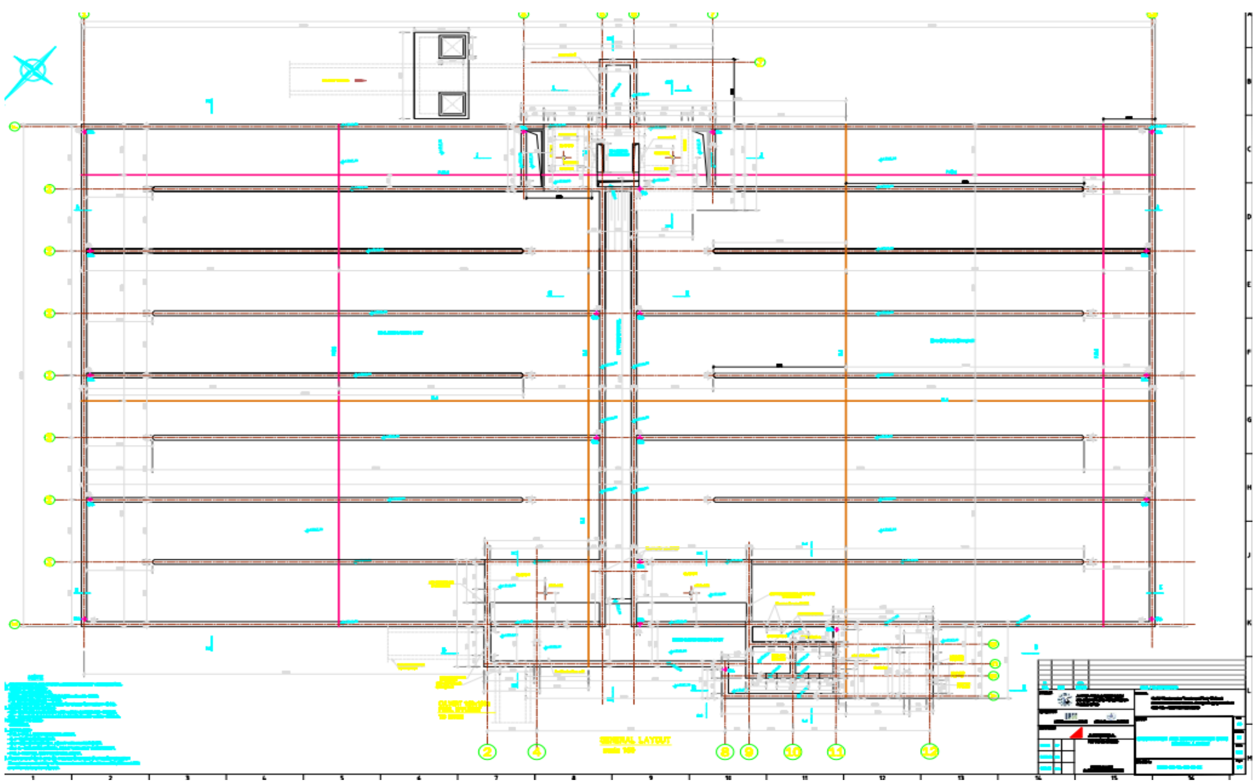


Fig 3.9 Chlorination Unit

3.2 Data collection

In order to achieve the objectives of the research stated in chapter one, two types of data are required.

The methodologies implemented to collect the data are discussed below.

a. Sampling of soil up on which the tanks are constructed

The type of data required for this study will be the soil sample up on which the tanks are constructed. Therefore; representative sample of soils were taken carefully.

b. Design data of selected the tanks under the study:

The design data has vital role in understanding the source of the problem, and in proposing the right solution. This data is collected by having discussion with the Engineer.

3.2 Scenario Setting (Methods of analysis)

Data collected from sample shall be analyzed in laboratory and also the design data shall be studied to forecast the implication of the results from the analysis to the short and long term performance of the tanks. However; data collected from design are more of descriptive type and qualitative analysis method has been implemented.

Chapter 4: Results and Discussion

In this chapter; previous studies about property of expansive soils in Ethiopia & Addis Ababa and damage assessment & analysis for the hydraulic structures in question have been thoroughly discussed.

4.1 Previous studies about damage caused by expansive soil

Regarding damage caused by expansive soil a research was conducted in Addis Ababa by (Afewerk, 2004). In this study ninety-six randomly selected lightweight buildings founded on expansive soils of Addis Ababa were examined and detail study of one selected building was also conducted. The damages were related to the observational data and systematically analyzed and causes of damages were attributed to wall construction material, foundation system, drainage condition etc. Out of the observed ninety-six buildings, sixty-nines (72%) are affected by consequence of heave or shrinkage of the expansive soil. The problems observed on these 96 buildings vary from simple crack on walls to severe structural damages, which cause collapse of the whole building.

Research has also been done by Selamawit Mekbib for Sebeta City (2015 AAiT) on active areas for new building construction. Specific gravity, particle size distribution, Atterberg limits and free swell ; unconfined compression strength and one dimensional oedometer tests of the soil were conducted. Based on laboratory test results and site observation the soils in the town were grouped in to five categories. From the test results it is concluded that some part of the town is covered by expansive soils especially around rolling areas. Yet, a small part of the town is rocky. Besides, the findings indicated that around depth of 1.5m 10% MH, 31% CH, 39% Expansive and 20% Rock whereas 18% MH, 19% Expansive, 33% CH, 10% CL and 20% Rock at depth of 3m were found. Moreover majority of the built up area have soil of high plasticity and great amount of clay particles.

Another study by Tibebu Solomon 2016 (AAiT) has tried to assess the extent of damages caused by expansive soil on buildings constructed in Bahir Dar. Seventy Eight buildings were randomly selected from 20 different expansive soil locations. The assessment showed that out of those 78 buildings 68(87%)

of them are adversely affected. In most buildings more than one building components are damaged. The damage has been analyzed and interpreted systematically with respect to type of foundation, wall, floor, pavement, door, window and fence. The extent of damage is very high on pavements and decreases when we proceed to walls, floors and foundation in their respective orders. The study showed that the main cause of failure is fluctuation of ground water table. In addition to this poor surface drainage, presence of tress and vegetation close to buildings, damaged utility lines and improper design also have great contributions. Based on the causes of failures and other factors attempt was made to propose remedial measures for already damaged buildings constructed in expansive soil locations of the study area. In addition to this suitable foundation types and construction details have been also proposed for new buildings to be constructed in these areas.

Study has been done by Daniel Tekelu (2003 AAiT) to examine the swelling properties and pressure of Addis Ababa expansive soil.

4.3 Structural data analysis

4.2.1 General

Kality Waste Water Treatment Plant-Civil and Electro-Mechanical Works- Design, Supply and

Built Project comprises of major reinforced concrete structure such as Secondary Clarifier which is components of the treatment system proposed to be built in Addis Ababa.

There are four units of structurally symmetrical Secondary clarifier tanks and two circular slabs mounted on four columns as shown. Only one unit has been considered for structural modeling and designing purpose. The circular slab -columns system has been modeled as structurally independent entity as shown

The structures are subjected to different types of loading with varying magnitude within its design period. Within this period, the structures shall withstand the loads, with tolerable damage and deliver services for which it is intended for.

So that it delivers its intended services, the structure has been designed to safely carry extreme load (ultimate load) with %5 probability of being exceeded .Moreover, it is designed to carry service loads that impart acceptable cracking width and deformations on to the structure.

Generally speaking, ultimate limit state of design approach has been followed to design the structures. Under limit state design approach, structure is designed for service as well extreme loads.

For the service loads, especial attention was paid to limiting the crack width of the structures. By satisfying the minimum requirement for the crack width, it is possible to limit permeability of concrete; as result, the durability of concrete would be ensured. Maximum crack width of 0.2mm has been set as maximum tolerable crack width under the service loads.

The responses of the structures to extreme loads and the service loads were determined with help of SAP2000 software with capability of static as well as dynamic finite element analysis.

The latest version, version 14, of the software was employed for the same purpose.

The responses obtained at critical sections of the structure, as result of the analysis, were used to determine the reinforcement and check adequacy of the components of the structures and calculate the crack width.

The determination and the checking were based on the recommendations of ACI 350-06, ACI 350.3.01, ACI 318-01, ASCE-7-01, BS8007, EUROCODE-8, EBCS-1, EBCS-8 and AASHTO. So far, there is no internationally recognized code for designing cylindrical treatment tank with conical base. Even, ACI do not provide detail recommendation for designing of pure conical tanks and cylindrical circular tank except referring to PCA (Portland Cement Association) recommendation for cylindrical circular tank (CCT) design. However, PCA recommend designing of conical tanks by converting their dimension to equivalent cylindrical circular tank and designing the former accordingly. For designing of this secondary clarifier of circular shape at the top and conical shape at the base the recommendation of PCA for conversion of the bottom conical shape to equivalent circular tank has been followed for purpose of analysis and design.

4.2.2 Design Inputs

Reports of geological, geotechnical and seismology studies provided inputs for structural analysis and subsequent design of the structures. In addition to these reports, drawings issued after hydraulic as well as mechanical design provided in puts for structural modeling of the structure.

4.2.3 Inputs from Hydraulic and Electromechanical Designs

The Secondary Clarifier has been designed hydraulically to be circular shape at top and conical shape bottom below which cylindrical Hooper exists.

The following information has been obtained from hydraulic or process design:-

- ✓ Maximum water level=2150.9m
- ✓ Top level of circular part=2151.48, internal dia=32m.
- ✓ Bottom level of Circular part=2148.4,
- ✓ Top level of conical part=2148.4,internal dia.=32m
- ✓ Bottom level of Conical part=2147.20,internal dia.=4.7m
- ✓ Top level of Hooper=2147.2,internal dia.=4.7m
- ✓ Bottom level of Hooper(top slab level)=2144.36,internal dia.=1.87m
- ✓ Weight of Scrapper=about 3890KG

4.2.4 Inputs from Geotechnical Investigations

Geotechnical investigations and the observation after foundation excavation revealed that black cotton soil exists below foundation slab levels. As per recommendation of the geotechnical investigation, the black cotton soil will be removed and backfilled with another material (back fill foundation material) with better performance in terms of expansiveness. Tests have been carried out to determine engineering properties of in-situ-black cotton and backfill foundation materials. The following values of engineering properties of the materials were used as input for designing purpose of the secondary clarifier.

A) Design Engineering Properties of Black Cotton Soil (insitu-soil)

- Allowable bearing capacity=285Kpa.
- Sub-grade reaction=34MN/m3.

The bearing capacity of the foundation material has been estimated based on the relation based on different authorities; Therefore, range of values have been mentioned in report is as representative values. However, a single result has been picked for designing pupose.

Sub-grade reaction of backfill material has been estimated based on the bearing capacity, based on the following relation (Bowles, 1997).

K_s=40$\times SF \times q_a$ where,

K_s is modulus of sub grade (Mpa.),

q_{ult} is ultimate bearing capacity(Kpa.),

q_a is allowable bearing capacity(Kpa.), and

$SF = q_{ult} / q_a$

Based on the following recommendation of Geotechnical report, the design parameters has been adopted to simulate structure-foundation interaction.

Even though values of engineering properties (magnitudes of bearing capacity and sub-grade reaction) for both in-situ and backfill soils are available ,the values of the in-situ soil ,which are less than that of backfill soil, ,have been used to model foundation-structure interaction. In addition, has been assumed that there is no potential of liquefaction of the backfill foundation material since the ground water level is significantly below foundation slab level of the structure.

4.2.5 Inputs from Architectural (Mechanical) Drawings

Civil drawings or Mechanical drawings, the result of after the hydraulic or process design or electro-mechanical design, were used for:-

- Structural modeling **,**
- Determination of self-weight of , and
- Fixing the original ground levels and foundation level, height of water.
- Preparation of formwork drawings,

4.2.6 Inputs from Seismic Hazard Analysis

The study conducted on the seismicity of the region in which the project located revealed that structures of the project may be exposed to standard horizontal peak ground acceleration as big as 0.05 g. As per geological study, it is located in moderate seismic risk zone. The risk can be projected with help of seismic hazard map. For that matter, Ethiopian building seismic hazard map given in seismic code (EBCS-8), has been employed to represent the risk in form of peak ground acceleration. It recommends to peak ground acceleration of 0.05g to be used for building in such area as Kaliti. Even though the map been prepared to predict peak ground acceleration for building, it has been used to forecast the peak ground acceleration for the treatment facilities as per the request of the client.

4.2.7 Specification of Construction Materials

Construction material properties of reinforced concrete structures were specified in view of ensuring their durability and strength so that they can withstand loads and adverse environmental conditions with tolerable damage; and carry out their intended functions properly during their design life.

Next to specification of the construction materials, during analysis stage ,it was assumed that the concrete of the structures will not be in contact with harmful substances that exist in foundation material and contained liquid; moreover, it was envisaged that the concrete will be impermeable enough not to allow migration of little waste water or other liquid resulting from chemical reaction in the tank.

To impart the impermeability to the concrete, its grade has been selected to be 30Mpa. as characteristic cubic strength of the concrete.

- The properties of the concrete and re-bar specified for static analyses of the structures:-
  - Characteristic cylindrical compressive strength of structural concrete (f'c)=24 Mpa..
  - Maximum allowable Tensile stress in concrete (0.1* f'c)=2.4Mpa.
  - Characteristic yield strength of reinforcement bars (fy)=500 Mpa. for all bars that carry flexural as well as shear stresses
  - Poisson's ratio for the structural concrete =0.2
  - Coefficient of shrinkage for reinforced concrete=0.0003
  - Modulus of Elasticity of concrete (Ec) = 28.98Gpa.
  - Modulus of Elasticity of steel (Es)=200Gpa.

For purpose of analysis, the concrete sections of the structures have been assumed not to be cracked regardless of level of stress sustained by the concrete section. Consequently, the seismic as well as the static analysis of the structure were presumed to e elastic in nature. The same nature will exist for calculation of crack width instigated by service loads. Nevertheless, designing of the section for flexure and shear has been based on cracked section.

4.2.8 Loads considered for design

- ✓ Self-weight of the Secondary Clarifier
- ✓ Hydrostatic load
- ✓ Equipment Loads
- ✓ Load due temperature change
- ✓ Seismic inertial load
- ✓ Hydrodynamic Load

4.2.8.1 Hydrostatic Load

The hydrostatic loads were calculated as linearly distributed loads on respective walls and the unit weight of the waste water has been taken to be 10KN/m^3.For estimation of hydrostatic force , full water level which is at equal level to top elevation of tank has been taken. The hydrostatic load has been directly applied onto the 3-D finite element model.

4.2.8.2 Earth Pressure Load

No significant earth pressure is expected to acts on walls of Secondary Clarifier.

4.3 Geotechnical analysis of the secondary clarifiers

The maximum bearing pressure at the base of the foundation slab below the peripheral wall is found to be about 130Kpa, see figure-20. The induced foundation pressure is well below the allowable pressure of the back filled foundation material (285Kpa.)

Table.1. Flexural Design of Re-bar for Walls, and Foundation Slab

Location of critical section	Mu (KN m/m)	h (mm)	d (mm)	b (mm)	a (mm)	ρ	As Required (mm2)/m	As Provided (mm2)/m	Remark
Peripheral Walls	23	300	245	1000	5.16	0.00086	210.84	523.33 (Φ10c/c150)	Vertical bars
Foundation slab	23	250	195	1000	6.53	0.00137	266.57	638.47 (Φ12c/c300+ Φ10c/c300)	Radial Bars
Hooper conical walls	36	300	245	1000	8.138	0.00135	332.045	565.2 (Φ12c/c200)	Inclined bars

Table.2. Computation of Crack width due service load

Foundation slab	30	250	195	1000	118.7	It is ok
Hooper conical walls	30	300	245	1000	127	It is ok

Settlement calculation

As can be seen from the geotechnical laboratory results of the backfilling soil the soil is expansive which in fact is susceptible to primary and secondary settlement. Practically it is impossible to prevent settlement of shallow foundations. At least, elastic settlement will occur.

There are three types of foundation settlements

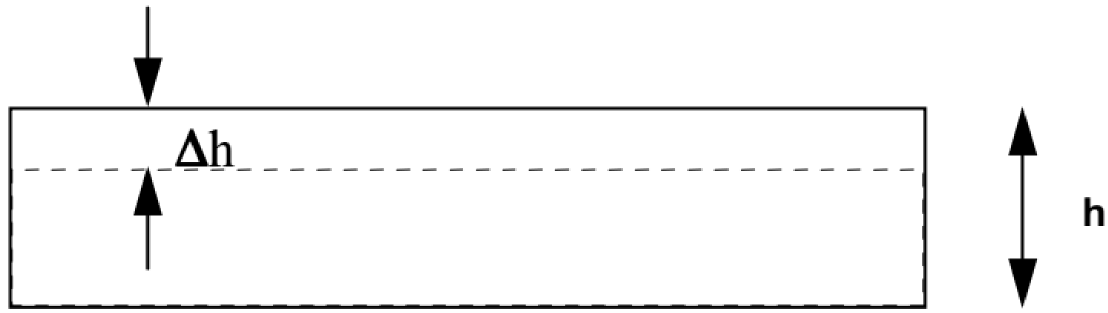
1. Uniform settlement
2. Tilt or distortion settlement
3. No uniform or differential settlement

However since the tanks are founded on mat foundation of reasonably high thickness (30cm at the walls) and the tanks are mainly subjected to hydrostatic pressure load which is uniform so that it is reasonable to expect no differential settlement for a single tank. But since the tanks are constructed at different time for different duration and the tanks may be filled at different period, differential settlement among the tanks can be expected.

Immediate settlement

$$\rho e = \frac{P}{EuL}(1-vu^2)\mu_s\mu_{emb}\mu_{wall}$$

Settlement of a Single Infinite Layer



$$\frac{\Delta hy}{\mathrm{h}} = \frac{e_{f-}e_i}{1 - e_i}$$

Where e_i and e_f are the initial and final voids ratio

$$\Delta h = h\frac{e_{f-}e_i}{1 - e_i}$$

The bottom level of the tanks is set to be at elevation of 2144.06 above sea level

It is observed that the subsurface condition is changed to basaltic rock overlaid by the thick layered soil at the level of 2148.88 above sea level.

4.3.1 Structural models the secondary clarifier and the response of the foundation to the imposed loads

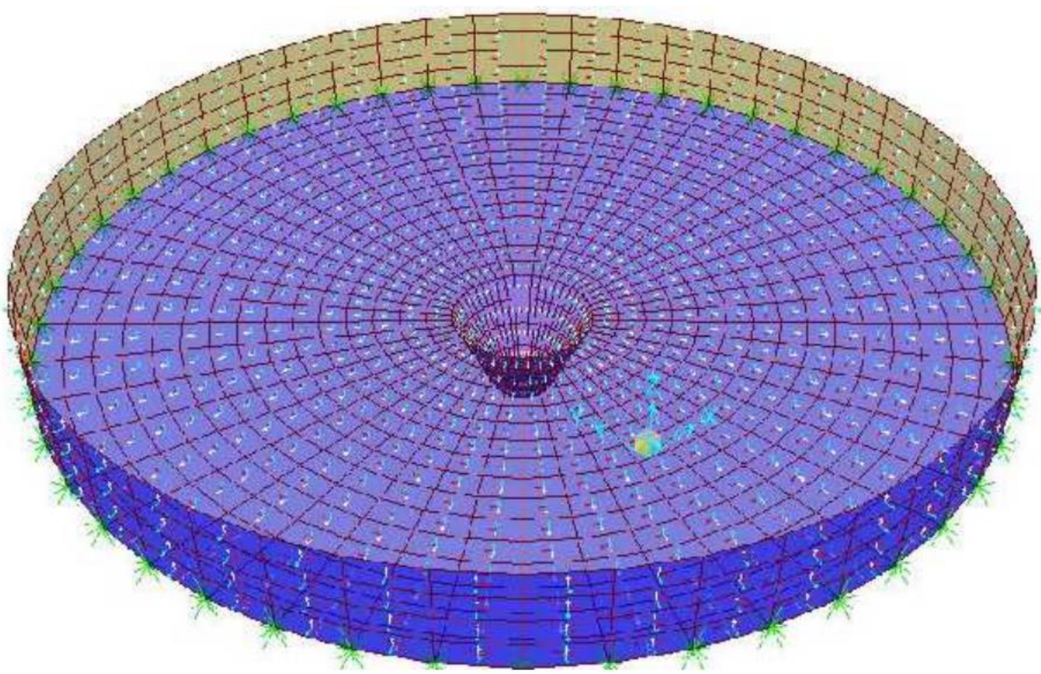


Fig 4.1:3-D Model of Secondary Clarifier (AKTOR Technical Societe', 2015)

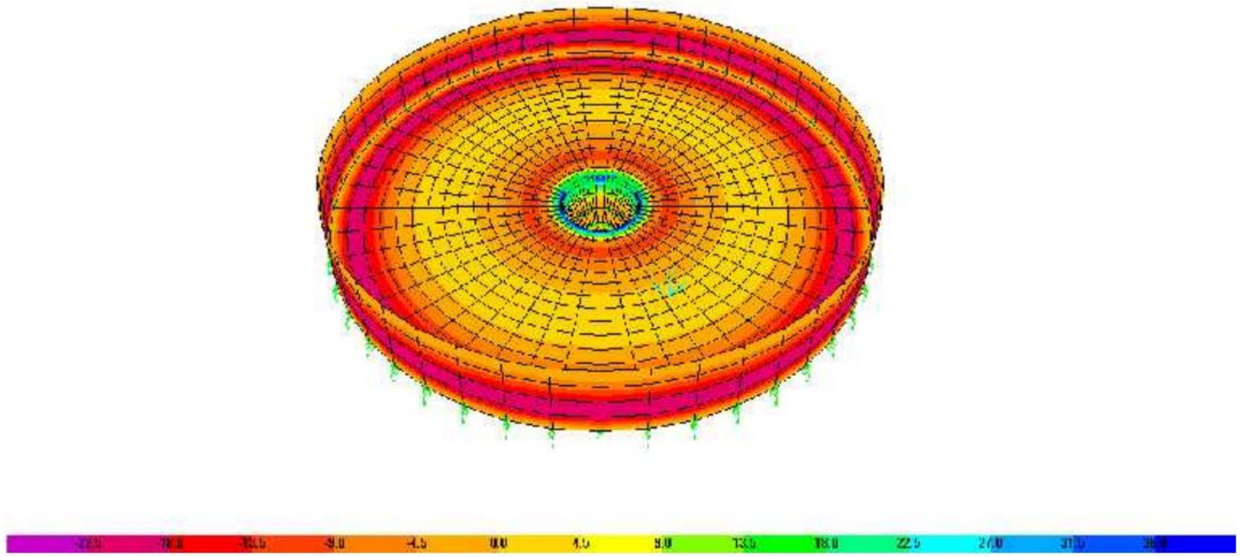


Fig 4.2: Wall and foundation slab of Secondary Clarifier: Bending Moment (AKTOR Technical Societe', 2015)

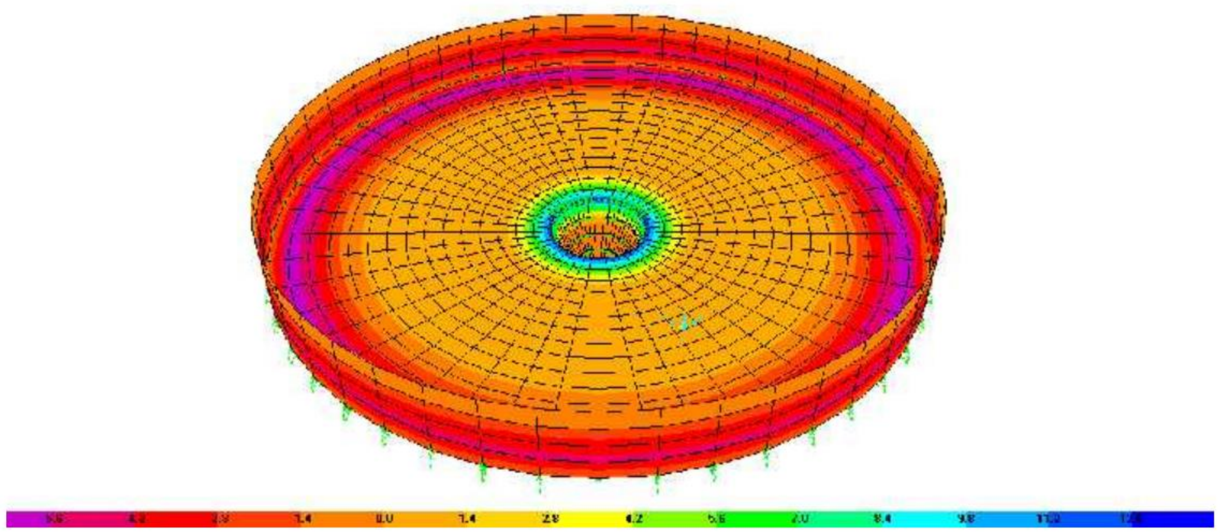


Fig 4.3: Wall and foundation slab of Secondary Clarifier: Bending Moment (AKTOR Technical Societe', 2015)

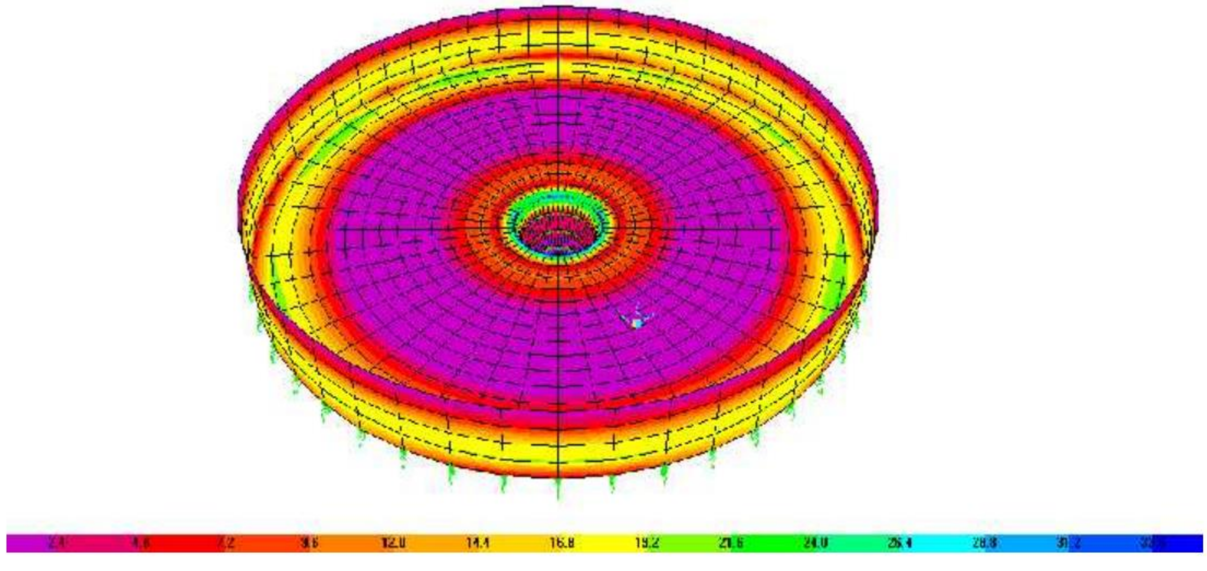


Fig 4.4: Wall and foundation slab of Secondary Clarifier: Bending Moment (AKTOR Technical Societe', 2015)

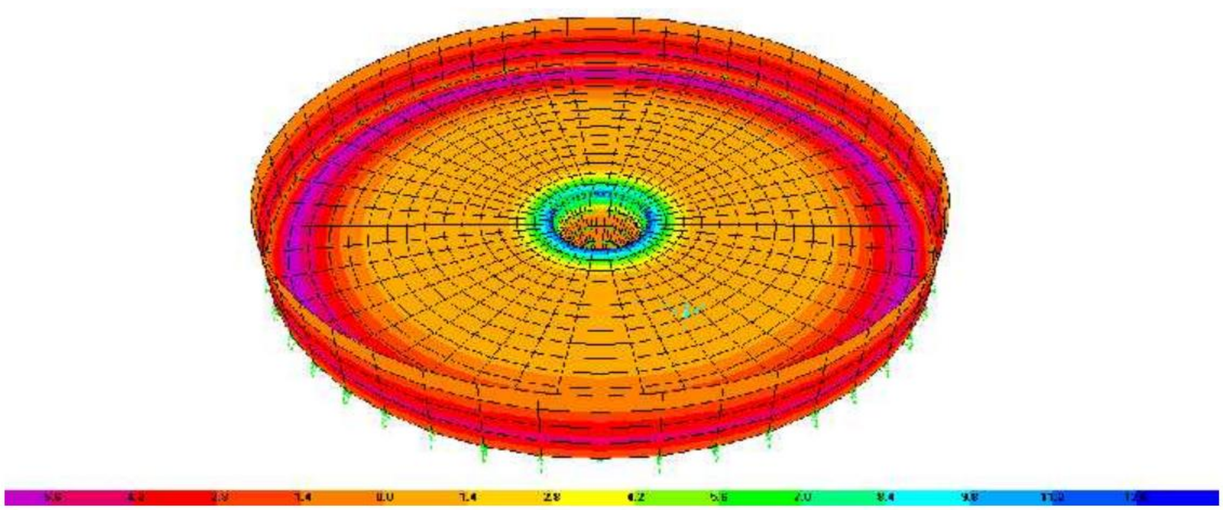


Fig 4.5: Wall and foundation slab of Secondary Clarifier: Bending Moment (AKTOR Technical Societe', 2015)

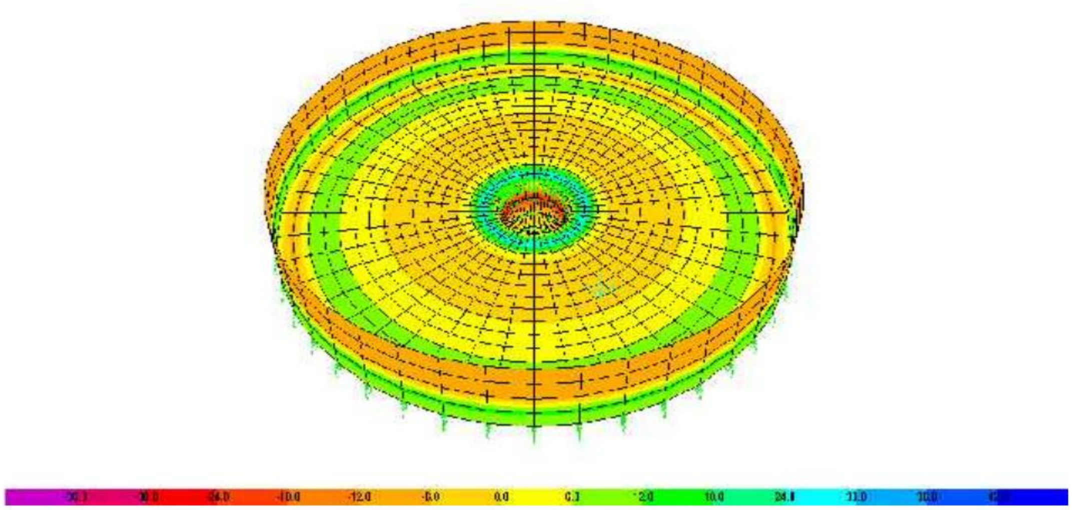


Fig 4.6: Wall and foundation slab of Secondary Clarifier: Shear force (AKTOR Technical Societe', 2015)

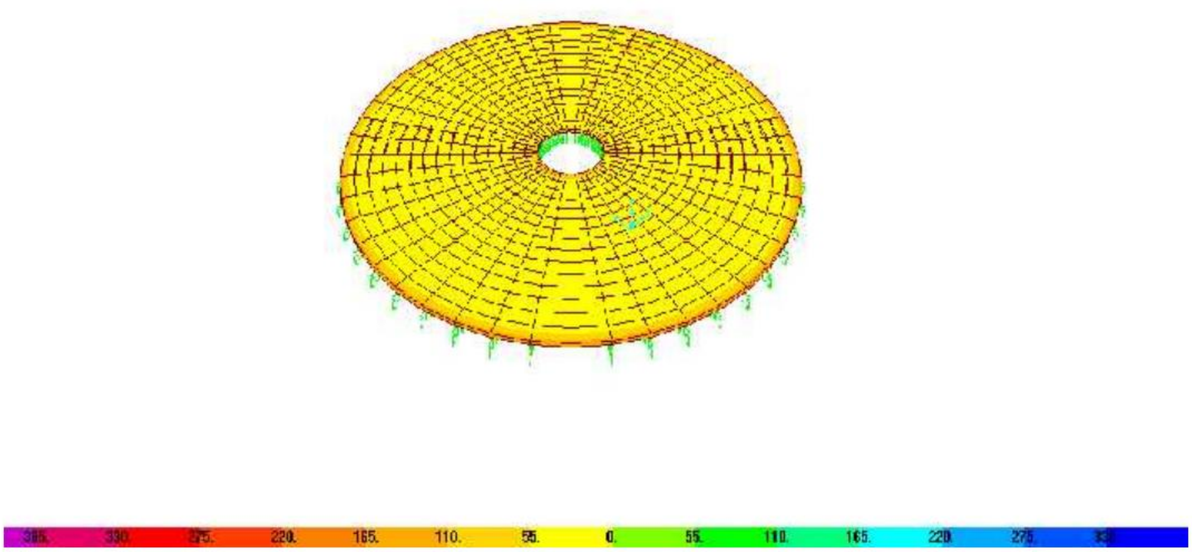


Fig 4.7: Secondary Clarifier foundation Slab: Bearing Pressure (KN/m2) (AKTOR Technical Societe', 2015)

As it has been shown from the above geotechnical and structural data and analyses the tanks under consideration are founded up on the soil which is expansive. Any structure constructed up on such type of soil material would be susceptible to differential heaving and settlement.

However, even though the soil material nature is expansive and questionable in bearing capacity the compaction is good and the uniformity of the quality is the same throughout a single tank span. In this kind of situation if there is any settlement within a single soil-foundation interaction it is expected to uniform throughout the span.

But there would be four different scenarios in which differential settlement and heaving will damage the structures and the whole plant in general as can be tried to show as follows

1. When a single tank is filled with water and the other three tanks are empty

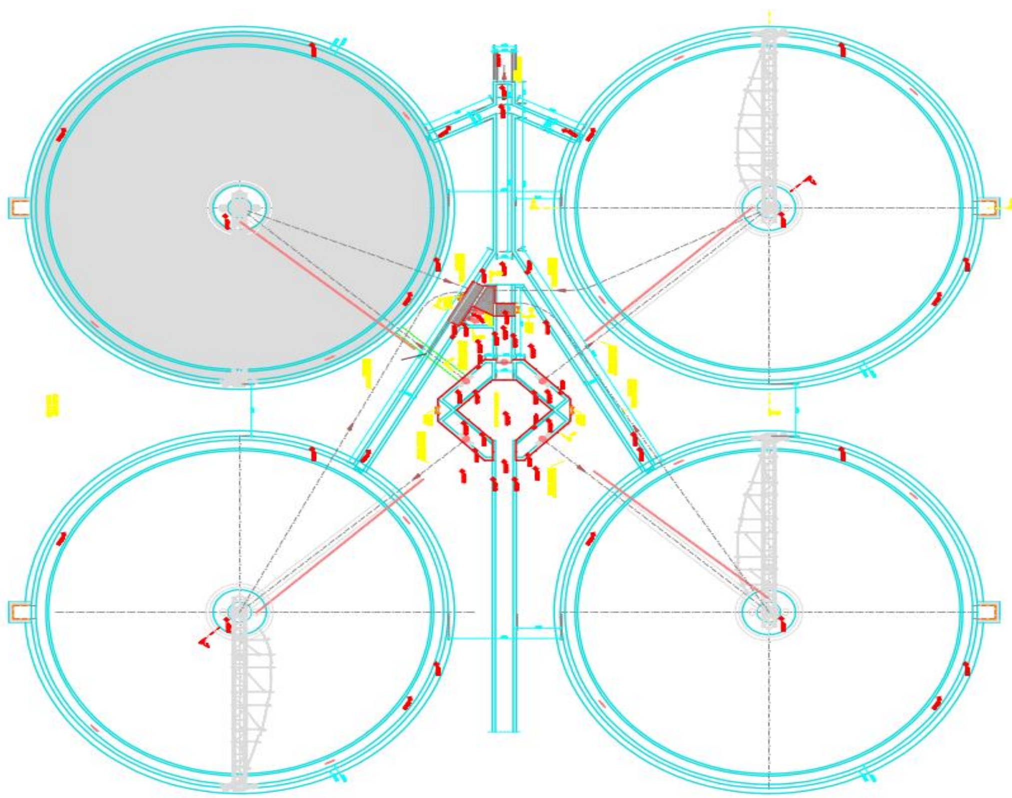


Fig 4.8: Case I (single tank)

The soil under the full tank is more stressed than the other three empty tanks where the expected settlement would be greater than the soil under the less stressed soil under the empty tank. Thus it is clear that the settlement is surely differential. This differential settlement will have a negative impact in the operational performance of the tanks.

2. <u>When the two alternatively opposite tanks are filled water and the other two are empty</u>

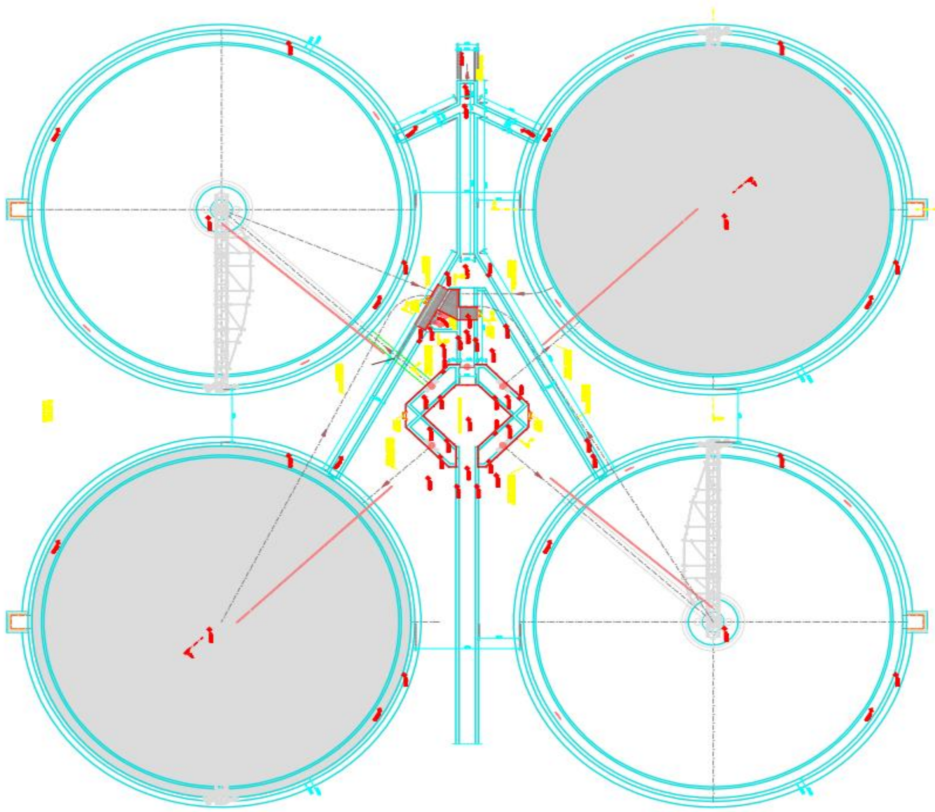


Fig 4.9: Case II (Two tanks filed)

The soil under the full tanks is more stressed than the other three empty tanks where the expected settlement would be greater than the soil under the less stressed soil under the empty tanks. Thus it is clear that the settlement is surely differential. Thus the settlement will disrupt the operation of the tanks.

3. <u>When the three tanks are filled with water and a tank is empty</u>

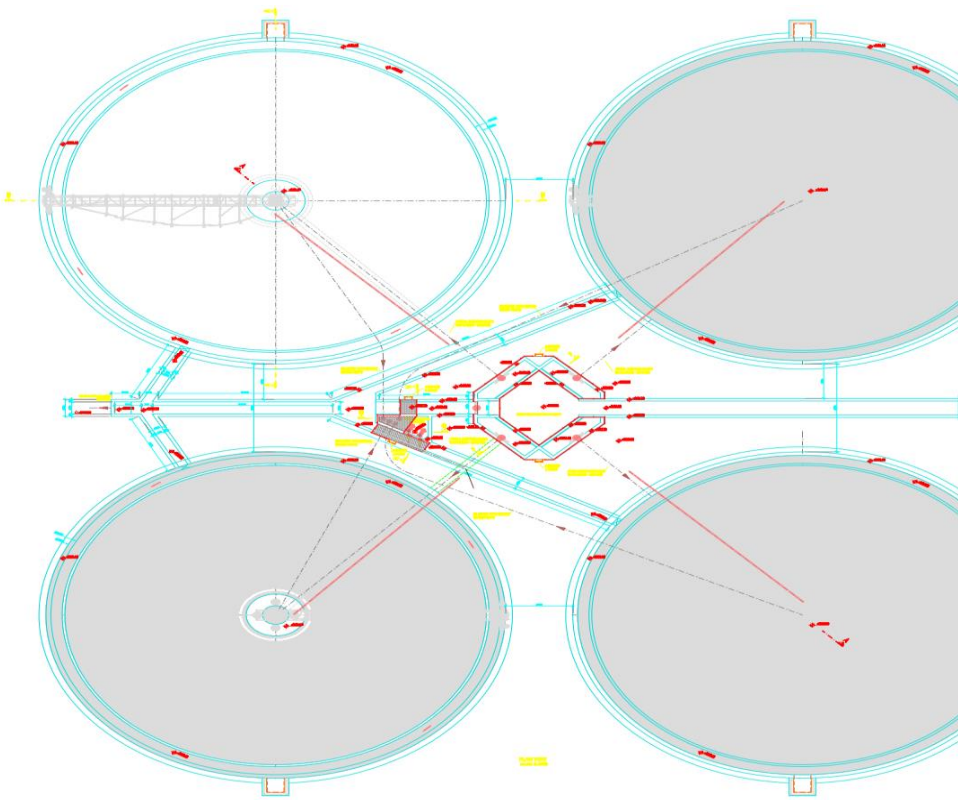


Fig 4.10: Case III (Three tanks filed)

The soil under the full tanks is more stressed than the other three empty tanks where the expected settlement would be greater than the soil under the less stressed soil under the empty tanks. Thus it is clear that the settlement is surely differential. Thus the settlement will disrupt the operation of the tanks.

4. When all are filled water

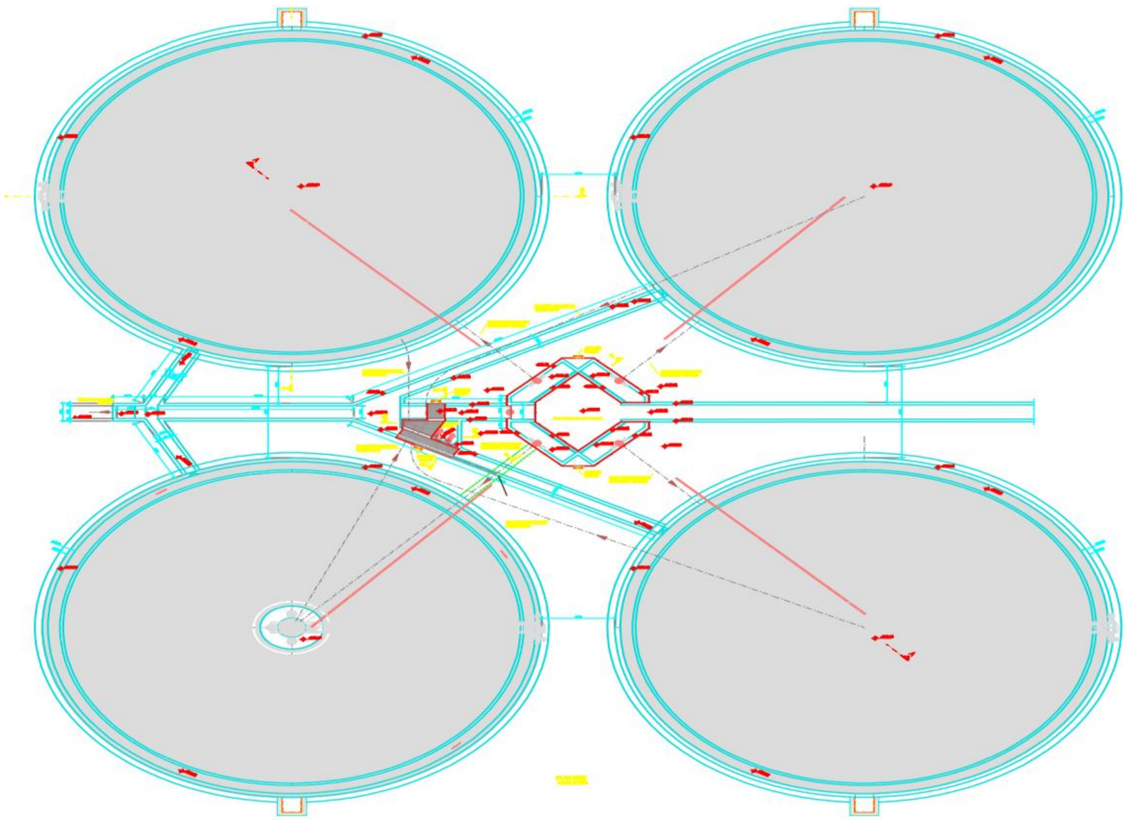


Fig 4.11: Case IV (Four tanks filed)

The soil under the full tanks is stressed equally so that there would not be differential settlement or heaving in this case. Thus the performance of the tanks is also may not be compromised. But the hydraulic gradient with the upstream and downstream structures would not be the same as with the one which is proposed in the design.

Chapter 5: Conclusion and Recommendation

5.1 Conclusion

1. The subgrade nature is varying in property and strength throughout the plant. Majority of the area is covered with basaltic rocky subsurface which is very suited for construction of strong foundation.

2. The subgrade under the considered tanks is black cotton soil which is highly expansive.

3. The soil under a single tank is most likely to heave or settle uniformly.

4. Alternative filling or emptying of the tanks results in differential stress then differential settlement among the tanks in question.

5. Since the flow is gravity driven any settlement or heaving in any one of the tanks within the hydraulic gradient will alter the whole pattern of the flow. Altering the flow pattern then creates unprecedented problems on the performance of the plant. It may also lead to the overall failure of the plant in a bigger scale.

5.2 Recommendation

To prevent the expected failures complications above; the following recommendations may help

1. The construction procedure should strictly follow the standard methodology i,e the expansive black cotton should be removed and replaced and compacted properly.
2. The design also should consider the actual condition and should also provide a factor of safety for such improper construction procedures
3. Since the tanks are already constructed ; the settlement or heaving may be prevented in different methods such as:
   - Filling all the tanks at the same time and at the same rate
   - Constructing the discharging culverts after the maximum settlements are achieved first by filling all the tanks to exert the hydrostatic pressure
   - Providing a cut-off trench around the tanks to intercept any subsurface water not to wet the expansive soil
   - Providing an impermeable high strength concrete to prevent penetration of water to the foundation soil

References

Afewerk. (2004).

Ahmed.M.E. (20011). Foundations on Expansive Soils: Sudan Experience. *Building and Road Research Institute, University of Khartoum.*

AKTOR Technical Societe'. (2015).

Bani-Han, M. a. (2007). Cracking of RC School Building Due to Soil Expansion. *Jordan Journal of Civil Engineering, Volume 1, No. 4*.

Leikun, T. &. (1999). Soil Mechanics.

Lucian.C,]. (2006). Geotechnical aspects of buildings on expansive soils in Kibaha, Tanzania.

Lulu. (2010). A simple technique for estimating the 1-D heave of natural expansive soils. *Department of Civil Engineering University of Ottawa.*

Mekbib, S. (n.d.).

Nelson. (1991). Expansive soils - problems and practice in foundation and pavement engineering. *Mille Dept. of civil engineering, Colorado University*.

Osman. (2005). The prediction of damage condition in regards to damage factor influence of light structures on expansive soils in Victoria, Australia.

Rogers.J.D. (n.d.). Damage to foundations from expansive soils.

Sharma. (1987). A Remedial measure for cracked buildings in expansive soils. *Central Building Research Institute, India.*

Selamawit Mekbib (2015 AAiT) Specific gravity, particle size distribution, Atterberg limits and free swell ; unconfined compression strength and one dimensional oedometer tests for soils in Sebeta City

Sissay. (2008). Assement of Damage of Buildings in expansive Soil areas of Addis Ababa. *Unpblished Msc Thesis*.

Tefera. (2008). *Addis Ababa University Pres*.

Tefera.A. (2008). Principle of Foundation Engineering. *Addis Ababa University press*.

Tessema.G., 1. (1984). Foundation on expansive soil. *Journal of EAEA Vol. 6*.

Tibebu Solomon 2016 (AAiT) damages caused by expansive soil on buildings constructed in Bahir Dar

Venkataramana, L. (2003). Building on Expansive Clay with Special Reference to Trinidad. *West Indian Journal of Engineering Vol. 25*.

Zewdie.A. (2004). Investigation into shear strength characteristics of expansive soil of Ethiopi. *Unpublished Msc thesis*.

Appendixes

1. Geotechnical Data

	የድርጅቱ ስም: Company Name: ሳባ ኢንጅነሪንግ ኃ/የተ/የግ/ማህበር SABA ENGINEERING PLC	Page No.: 1 of
		Effective Date: April 17, 2015
Issue No. 1	Document Title: Direct Shear test	Doc. Number: OF/MTL/SE/420

LAB. NO. : 1276/15
CLIENT : Actor Construction
PROJECT : Kality W.W.T.P
SAMPLED BY : Fill Material
SPECIEFIED BY : SABA Engineering ON :05/09/15
TEST ORDER SUBMITTED BY : SABA Engineering ON :05/09/15
TEST FOR : REMOLDED DI (AASHTO T236-84)
REPORTED TO : The Client ON :10/09/15

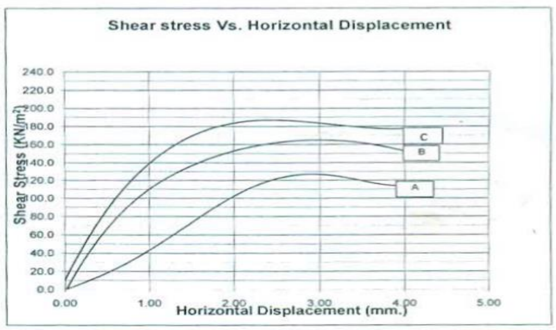
(Quick) REMOLDED DIRECT SHEAR TEST RESULTS

Sample ID Secondary Clarafiltter
Depth(m) -
Type of sample: Disturbed

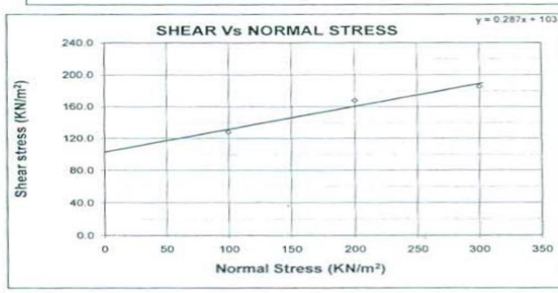
Bulk Density (gm/cm^3) : 1.668
Dry Density (gm/cm^3) : 1.361
Moisture content (%) : 22.6

Specimen size:
Diameter = 6cm.
Thickness = 3 cm.

Specimen	A	B	C
H.D(MM.)	shear str. (KN/m^2)	shear str. (KN/m^2)	shear str. (KN/m^2)
0.00	0.00	0.00	0.00
0.20	4.95	24.75	49.49
0.40	14.85	54.44	81.67
0.60	24.75	76.72	103.94
0.80	32.17	96.52	123.74
1.00	42.07	113.84	141.06
1.20	54.44	123.74	153.43
1.40	66.82	131.16	160.86
1.60	79.19	138.59	168.28
1.80	91.57	146.01	175.71
2.00	103.94	150.96	180.66
2.40	118.79	160.86	185.61
2.80	126.21	168.28	190.56
3.20	123.74	163.33	185.61
3.60	118.79	158.38	180.66
4.00	113.84	153.43	173.23



Speciemen	A	B	C
Normal Stress (KN/m^2)	100.00	200.00	300.00
Shear Stress (KN/m^2)	128.00	168.00	185.50



Cohesion, C (KN/m^2) = 103.00
Angle of internal friction, ϕ = 16.01

REMARKS:

REPORTED BY:
Lab. Engineer

APPROVED BY:
Ayehutsega Awgichew
Soil and Const. Mat. Testing Dept. Manager

	የድርጅቱ ስም: Company Name: ሳባ ኢንጅነሪንግ ኃ/የተ/የግ/ማህበር SABA ENGINEERING PLC	Page No.: 1 of Effective Date: April 17, 2015
Issue No. 1	Document Title: **Procto Density**	Doc. Number: OF/MTL/SE/415

LAB. NO.	:	1276/15	
CLIENT	:	Actor Construction	
PROJECT	:	Kality W.W.T.P	
SAMPLE SOURCE/STATION/CODE	:	Secondary Clara filter	
SAMPLE OF	:	Fill Material	
SAMPLED BY	:	The Client	
SAMPLE AND TEST ORDER SUBMITTED BY	:	The Client	ON : 05/09/15
SPECIEFIED BY	:	The Client	ON : 05/09/15
TEST FOR	:	Compaction (Modified AASHTO T180)	
TEST RESULT REPORTED TO	:	The Client	ON : 10/09/15

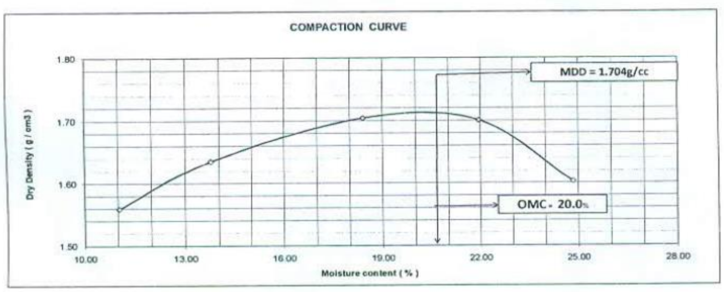
COMPACTION TEST RESULTS

(AASHTO DESIGNATION T- 180)

Trial No.	1	2	3	4	5
Wet soil (g)	1540	1655	1795	1845	1780
Wet density (g / cm3)	1.73	1.86	2.02	2.07	2.00

Moisture content determination

Wet soil (g)	171.00	182.00	193.00	200.00	206.00
Dry soil (g)	154.00	160.00	163.00	164.00	165.00
Moisture cont. (%)	11.04	13.75	18.40	21.95	24.85
Dry Density (g / cm3)	1.56	1.64	1.704	1.701	1.60



MDD(g/cm3) = 1.704

OMC (%) = 20.80

REMARKS :

REPORTED BY :

APPROVED BY :

Ayehutsege Awgachew
Soil and Const. Mat. Testing
Dept. Manager

COAQUATECH1

	Company Name:	
	SABA ENGINEERING PLC	Effective Date: April 17, 2015
Issue No. 1	Document Title: In-Place Density	Doc. Number: OF/MTL/SE/416

LAB. NO. : 826/15

CLIENT : AKTOR TECHNICAL SOCIETE ANONYME ETHIOPIA BRANCH

PROJECT : Kaliti Waste Water Treatment Plant

SAMPLE SOURCE/STATION/CODE : 5-A,7-B,9C

SAMPLE OF : Fill Material

SAMPLED BY : The Client

SAMPLE AND TEST ORDER SUBMITTED BY : The Client ON : 20/05/15

SPECIEFIED BY : The Client ON : 20/05/15

TEST RESULT REPORTED TO : The Client ON 21/05/15

IN - PLACE DENSITY DETERMINATION
SAND REPLACEMENT METHOD, AASHTO T191-86

UNIT WEIGHT OF SAND (As determined in the lab.), = 1.30

Density and moisture content Determination			
Sample code	5-A	7-B	9-C
Weight of soil in hole, gm	4070	4840	5170
Volume of hole, cm^3	2465	2792.3	3080.8
Field Bulk Density (gm/cm3)	1.65	1.73	1.68
Moisture content (M.C) in %	5.30	6.37	8.27
Field Dry Density(gm/cc)	1.57	1.63	1.55
% Compaction	96.5	100.3	95.4

Remark : Laboratory Maximum Dry Density(MDD) = 1.625g/cc

Reported by :

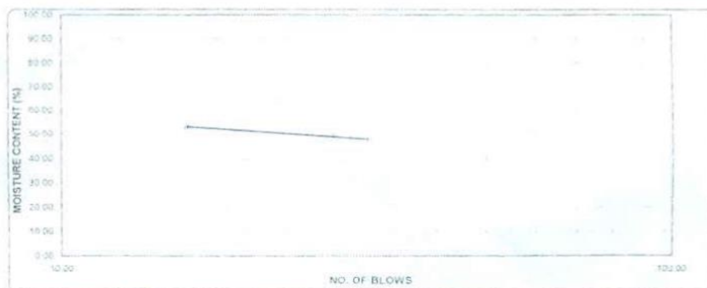
Approved by :

Document Title: Atterberg Limit , Liquid & Plastic Limit

LAB. NO.	:	790/15
CLIENT	:	Aktor Technicial socite Anonyme Ethiopia branch
PROJECT	:	Kaliti Waste Water Treatment Plant
SAMPLE SOURCE/STATION/CODE	:	stock pile at existing
SAMPLE OF	:	sub base material
SAMPLED BY	:	The Client
SAMPLE AND TEST ORDER SUBMITTED BY	:	The Client ON: 09/05/15
SPECIEFIED BY	:	The Client ON: 09/05/15
TEST FOR	:	Atterberg Limit(AASHTO T89,90-86)
TEST RESULT REPORTED TO	:	The Client ON: 21/05/15

TEST RESULT

	LIQUID LIMIT			PLASTIC LIMIT	
No. Blows	32	28	16		
Wt wet soil (g)	41.01	31.75	41.21	32.42	32.07
Wt dry soil (g)	30.33	[illegible]	34.46	31.22	31.36
Moisture content (%)	47.85	49.51	53.11	23.36	27.07
				AV. PL (%)	25.2



Liquid Limit	Plastic Limit	Plasticity Index	WET SEIVE ANALYSIS, % PASS			Unified soil Classification	AASHTO soil Classification
LL(%)	PL(%)	PI	2 mm	0.425 mm	0.075 mm		
50	25	25	30	23	17.0	GC	A-2-7(0)

REMARKS : Gravelly Clay

REPORTED BY :

APPROVED BY :

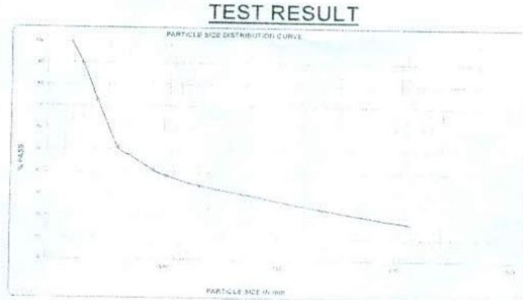
SABA ENGINEERING PLC

Wet & Dry Sieve Analysis

LAB. NO.	: 790/15
CLIENT	: Aktor Technical Socite Anonyme Ethiopia branch
PROJECT	: Kaliti Waste Water Treatment Plant
SAMPLE SOURCE/STATION/CODE	: Stock pile at existing
SAMPLE OF	: Sub base material
SAMPLED BY	: The Client
SPECIEFIED BY	: The Client ON : 9/5/2015
SAMPLE AND TEST ORDER SUBMITTED BY	: The Client ON : 9/5/2015
TEST FOR	: Wet sieve Analysis(AASHTO T27-84)
TEST RESULT REPORTED TO	: The Client ON : '21/05/15

TEST RESULT

SIEVE SIZE(MM)	% PASSED
63.50	100
50.00	89
37.50	73
25.00	51
19.50	47
12.50	41
9.50	38
4.75	34
2.00	30
0.425	23
0.075	17



REMARKS:

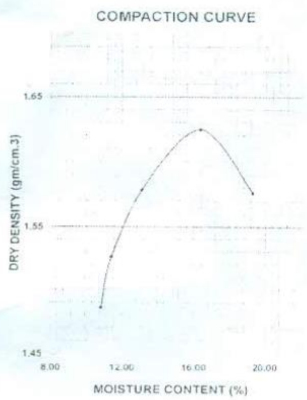
REPORTED BY :

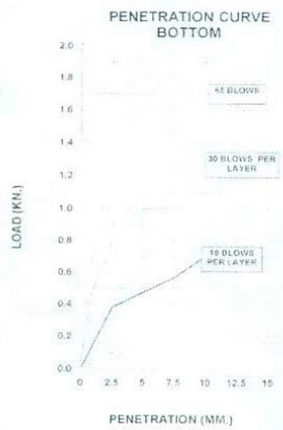
APPROVED By:

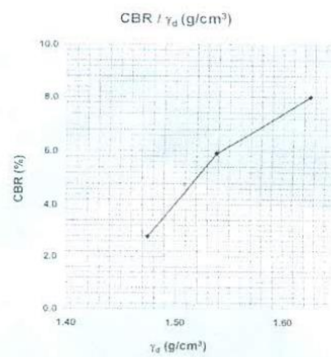
የድርጅቱ ስም፡ Company Name: ሳባ ኢንጅነሪንግ ኃ/የተ/የግ/ማህበር SABA ENGINEERING PLC		Page No 1 of
		Effective Date April 17 2015
Issue No 1	Document Title: Three Point CBR Proctor Compaction	Doc Number OF/MTL/SE/417

LAB. NO. : 790/15
CLIENT : Aktor Technical Socite Anonyme Ethiopia branch
PROJECT : Kaliti Waste water Treatment Plant
SAMPLE SOURCE/STATION/CODE : Stock pile at existing
SAMPLE OF : Sub base material
SAMPLED BY : The Client
SAMPLE AND TEST ORDER SUBMITTED BY : The Client ON: 09/05/15
SPECIFIED BY : The Client ON: 09/05/15
TEST FOR : Proctor Density and Three Point California Bearing Ratio,AASHTO T193,180
TEST RESULT REPORTED TO : The Client ON : 21/05/15

CALIFORNIA BEARING RATIO TEST RESULT
AASHTO T193-Three Point CBR - BOTTOM (Soaked)







CBR Value at 95% MDD = 6.2

AASHTO T180	
MDD gm/cm³	OMC (%)
1.625	16.50

BOTTOM Soaked

No. of Blows	Standard Load (Kn) 2.54mm	Standard Load (Kn) 5.08mm	Load(KN) 2.54mm	Load(KN) 5.08mm	CBR (%) 2.54mm	CBR (%) 5.08mm
10	13.61	20.41	0.38	0.47	2.8	2.3
30	13.61	20.41	0.80	0.99	5.9	4.9
65	13.61	20.41	1.09	1.28	8.0	6.3

CBR/Dry Density Data

NO Of Blows	γ_d g/(cm3)	CBR (%)	Swell (%)
10	1.48	2.8	1.37
30	1.54	5.9	0.84
65	1.63	8.0	0.78

Remark :

Reported by :

Lab. Engineer

Approved by :

Issue No	SABA ENGINEERING PLC Document Title: Los Angeles Abrasion	April 17, 2015 Doc. Number OF/MTL/SE/436
1		

LAB. NO.		826/15
CLIENT	:	AKTOR TECHNICAL SOCIETE ANONYME ETHIOPIA BRANCH
PROJECT	:	Kaliti Waste Water Treatment Plant
SAMPLE OF	:	Sub base Material
SAMPLED BY	:	The Client
SAMPLE AND TEST ORDER SUBMITTED BY	:	The Client ON : '20/05/15
SPECIEFIED BY	:	The Client ON : '20/05/15
TEST FOR	:	Los Angeles Abrasion
TEST RESULT REPORTED TO	:	The Client ON : '21/05/15

TEST RESULTS

Sr. No.	Sample Code	Material Type	AASHTO T96 - 83	
			Grading of Test Sample	LAA Loss (%)
1	-	Stock Pile at Existing Material	A	35

REMARKS:

REPORTED BY:

APPROVED BY:

LABORATORY

2. Structural Design and Analysis

2.1Gneral

Kality Waste Water Treatment Plant-Civil and Electro-Mechanical Works- Design, Supply and Built Project comprises of major reinforced concrete structure such as Secondary Clarifier which is components of the treatment system proposed to be built in Addis Ababa. The structure is subjected to different types of loading with varying magnitude within its design period. Within this period, the structures shall withstand the loads, with tolerable damage and deliver services for which it is intended for.
So that it delivers its intended services, the structure has been designed to safely carry extreme load(ultimate load) with %5 probability of being exceeded .Moreover, it is designed to carry service loads that impart acceptable cracking width and deformations on to the structure. Generally speaking, ultimate limit state of design approach has been followed to design the structures. Under limit state design approach, structure is designed for service as well extreme loads.

For the service loads, especial attention was paid to limiting the crack width of the structures.. By satisfying the minimum requirement for the crack width, it is possible to limit permeability of concrete; as result, the durability of concrete would be ensured. Maximum crack width of 0.2mm has been set as maximum tolerable crack width under the service loads. The responses of the structures to extreme loads and the service loads were determined with help of SAP2000 software with capability of static as well as dynamic finite element analysis. The latest version, version 14, of the software was employed for the same purpose. The responses obtained at critical sections of the structure, as result of the analysis, were used to determine the reinforcement and check adequacy of the components of the structures and calculate the crack width.

- The determination and the checking were based on the recommendations of ACI 350-06,ACI 350.3.01,ACI 318-01,ASCE-7-01,BS8007 ,EUROCODE-8,EBCS-1,EBCS-8 and AASHTO. So far ,there is no internationally recognized code for designing cylindrical treatment tank with conical base. Even, ACI do not provide detail recommendation for designing of pure conical tanks and cylindrical circular tank except referring to PCA (Portland Cement Association) recommendation for cylindrical circular tank (CCT) design. However, PCA recommend designing of conical tanks by converting their dimension to equivalent cylindrical circular tank and designing the former accordingly. For designing of this secondary clarifier of circular shape at the top and conical shape at the base the recommendation of PCA for conversion of the bottom conical shape to equivalent circular tank has been followed for purpose of analysis and design.

2.DEIGN INPUTS

- Reports of geological, geotechnical and seismology studies provided inputs for structural analysis and subsequent design of the structures. In addition to these reports, drawings issued after hydraulic as well as mechanical design provided in puts for structural modeling of the structure.

2.1. Inputs from Hydraulic and Electromechanical Designs

- The Secondary Clarifier has been designed hydraulically to be circular shape at top and conical shape bottom below which cylindrical Hooper exists.

The following information has been obtained from hydraulic or process design:-

- ✓ Maximum water level=2150.9m,
- ✓ Top level of circular part=2151.48,internal dia.=32m.
- ✓ Bottom level of Circular part=2148.4,
- ✓ Top level of conical part=2148.4,internal dia.=32m,
- ✓ Bottom level of Conical part=2147.20,internal dia.=4.7m

- ✓ Top level of Hooper=2147.2,internal dia.=4.7m
- ✓ Bottom level of Hooper(top slab level)=2144.36,internal dia.=1.87m
- ✓ Weight of Scrapper=about 3890KG

2.2 Inputs from Geotechnical Investigations

Geotechnical investigations and the observation after foundation excavation revealed that black cotton soil exists below foundation slab levels. As per recommendation of the geotechnical investigation, the black cotton soil will be removed and backfilled with another material (back fill foundation material) with better performance in terms of expansiveness. Tests have been carried out to determine engineering properties of in-situ-black cotton and backfill foundation materials.

The following values of engineering properties of the materials were used as input for designing purpose of the secondary clarifier.

A) Design Engineering Properties of Black Cotton Soil (insitu-soil)

- Allowable bearing capacity=285Kpa.
- Sub-grade reaction=34MN/m3.

The bearing capacity of the foundation material has been estimated based on the relation based on different authorities; Therefore, range of values have been mentioned in report is as representative values. However, a single result has been picked for designing purpose.

Sub-grade reaction of backfill material has been estimated based on the bearing capacity, based on the following relation (Bowles, 1997)

Ks=40*SF*qa where,

Ks is modulus of sub grade (Mpa.),

qult is ultimate bearing capacity(Kpa.), qa is allowable bearing capacity(Kpa.), and

SF = qult/ qa

Based on the following recommendation of Geotechnical report, the design parameters has been adopted to simulate structure-foundation interaction:-

Even though values of engineering properties (magnitudes of bearing capacity and sub-grade reaction) for both in-situ and backfill soils are available, the values of the in-situ soil ,which are less than that of backfill soil, ,have been used to model foundation-structure interaction.

In addition, has been assumed that there is no potential of liquefaction of the backfill foundation material since the ground water level is significantly below foundation slab level of the structure.

2.3 In puts from Architectural (Mechanical) Drawings

Civil drawings or Mechanical drawings, the result of after the hydraulic or process design or

electro-mechanical design, were used for:-

- **Structural modeling ,**
- **Determination of self-weight**
- **Fixing the original ground levels and foundation level, height of water.**
- **Preparation of formwork drawings**

2.4 Inputs from Seismic Hazard Analysis

The study conducted on the seismicity of the region in which the project located revealed that structures of the project may be exposed to standard horizontal peak ground acceleration as big as 0.05 g. As per geological study, it is located in moderate seismic risk zone. The risk can be projected with help of seismic hazard map. For that matter, Ethiopian building seismic hazard map given in seismic code (EBCS-8) ,has been employed to represent the risk in form of peak ground acceleration. It recommends to peak ground acceleration of 0.05g to be used for building in such area as Kaliti. Even though the map been prepared to predict peak ground acceleration for building, it has been used to foretell the peak ground acceleration for the treatment facilities as per the request of the client.

2.5 Specification of Construction Materials

Construction material properties of reinforced concrete structures were specified in view oensuring their durability and strength so that they can withstand loads and adverse environmental conditions with tolerable damage; and carry out their intended functions properly during their design life.

Next to specification of the construction materials, during analysis stage ,it was assumed that the concrete of the structures will not be in contact with harmful substances that exist in foundation material and contained liquid; moreover, it was envisaged that the concrete will be impermeable enough not to allow migration of little waste water or other liquid resulting from chemical reaction in the tank.

To impart the impermeability to the concrete, its grade has been selected to be 30Mpa. As characteristic cubic strength of the concrete.

- Characteristic cylindrical compressive strength of structural concrete (f'c)=24 Mpa..
- Maximum allowable Tensile stress in concrete (0.1* f'c)=2.4Mpa.
- Characteristic yield strength of reinforcement bars (fy=500 Mpa.
- for all bars that carry flexural as well as shear stresses
- Poisson's ratio for the structural concrete =0.2
- Coefficient of shrinkage for reinforced concrete=0.0003
- Modulus of Elasticity of concrete (Ec) = 28.98Gpa.
- Modulus of Elasticity of steel (Es)=200Gpa.

For purpose of analysis, the concrete sections of the structures have been assumed to be uncracked regardless of level of stress sustained by the concrete section. Consequently, the seismic as well as the static analysis of the structure were presumed to e elastic in nature. The same nature will exist for calculation of crack width instigated by service loads. Nevertheless, designing of the section for flexure and shear has been based on cracked section.

3. STRUCTURAL OBJECT MODELING

Secondary Clarifier has been structurally idealized with help of 3-D shell elements. This kind of model will help to idealize forces with 3-D spatial distribution and their interactions. The loads acting on to the tank has 3-D special distribution and they inter-act each other, see figure-4.

Shell element has been utilized to model the structure for purpose obtaining loads essential for designing. Using the elements, the 3-D modeling of the structures has been conducted.

The structural object modeling has been accomplished using SAP2000 Version 14.0.0 software which is with finite element modeling capabilities.

For sake of computation, the configuration of structure has been simplified to make it easy and on constructible geometry for modeling; Non-structural parts of structure have been removed just for purpose of modeling. For instance, slab-column system has not been part of the 3-D, however the loads coming from the system has been applied on the model as ‘‘external load’’. The external loads has been determined using the models of the system that portraits it as though it is independent entity, see figure-5.

The u-channel actually existing in the internal part of perimeter wall has not been represented in the 3-D model for sake of simplicity, since its contribution to total shear forces and bending moments are negligible. In other words, its contribution to total re-bar of the wall is negligible.

The geological investigation revealed that, the Secondary Clarifier is to be placed on backfill foundation that replaces black cotton soil up to depth of about 3m. The foundation-structure interaction has been simulated by using springs at attached at base of the foundation slab. The foundation stiffness has been represented by sub-grade reaction. The sub-grade reactions at portion where there is back fill has been taken to be 34MN/m3.

Figure-1upto Figure-3 below have been presented for purpose of providing geometrical information used for the modeling of the structure. The 3-D Model is shown in Figures below;

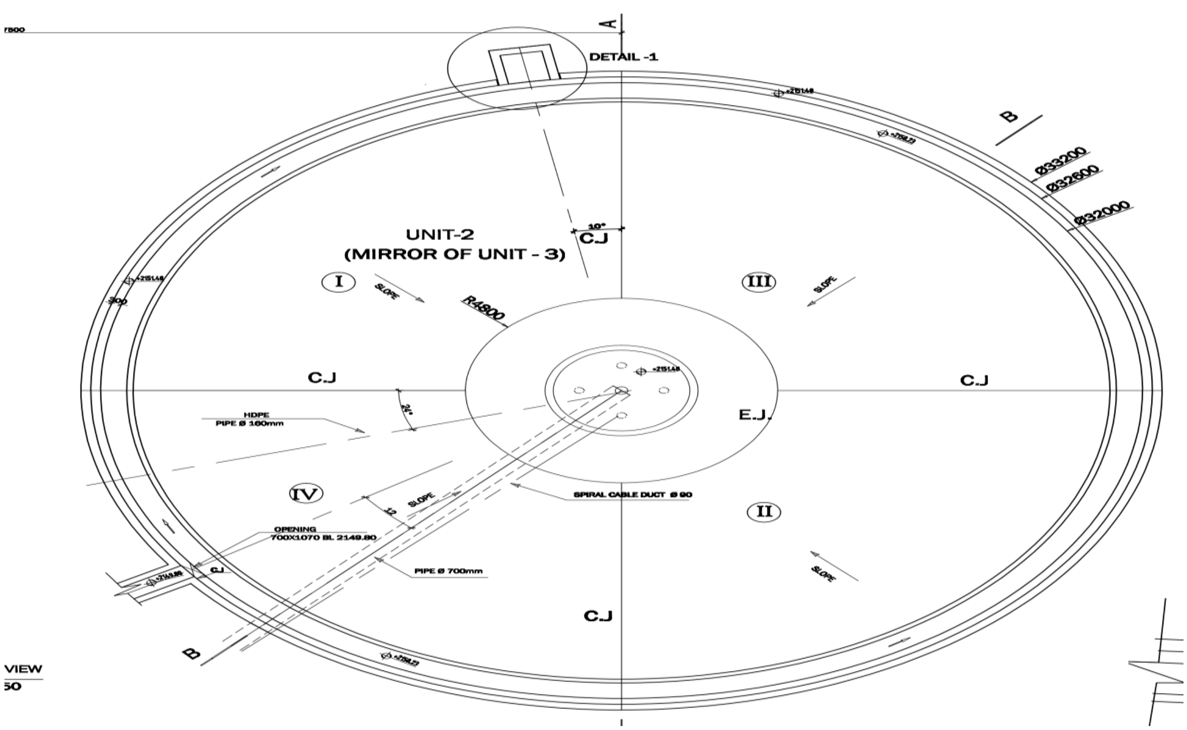


Figure-1: Plan of Single Unit

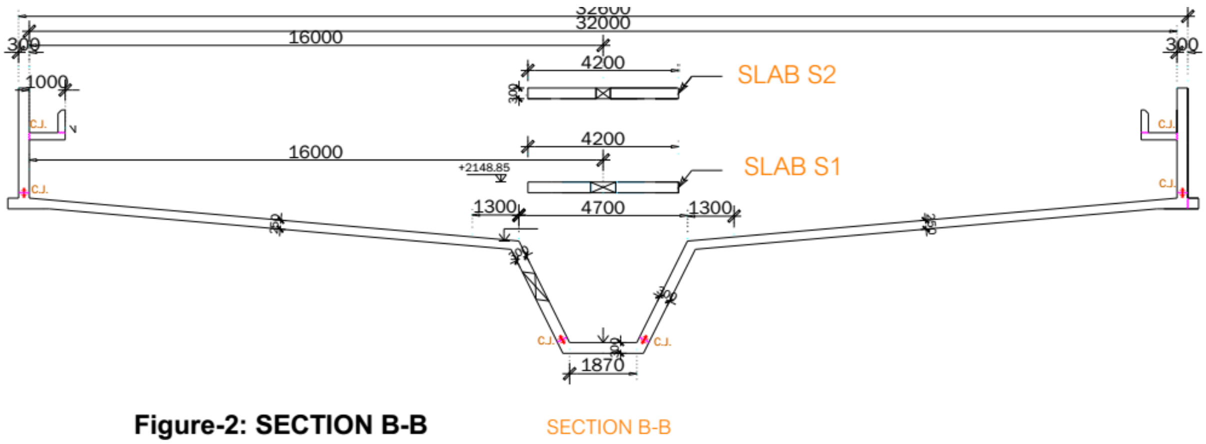


Figure-2: SECTION B-B

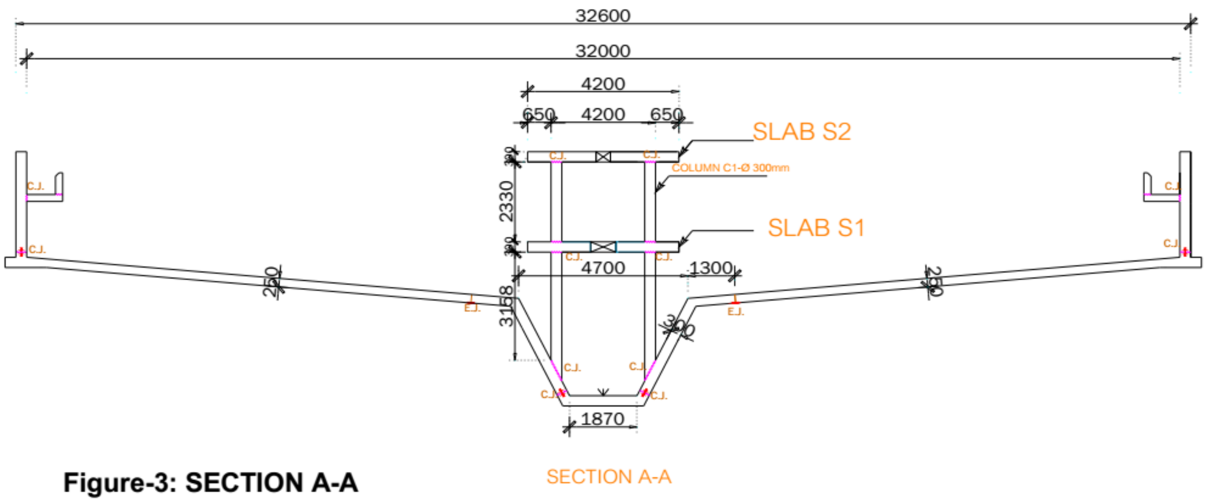


Figure-3: SECTION A-A

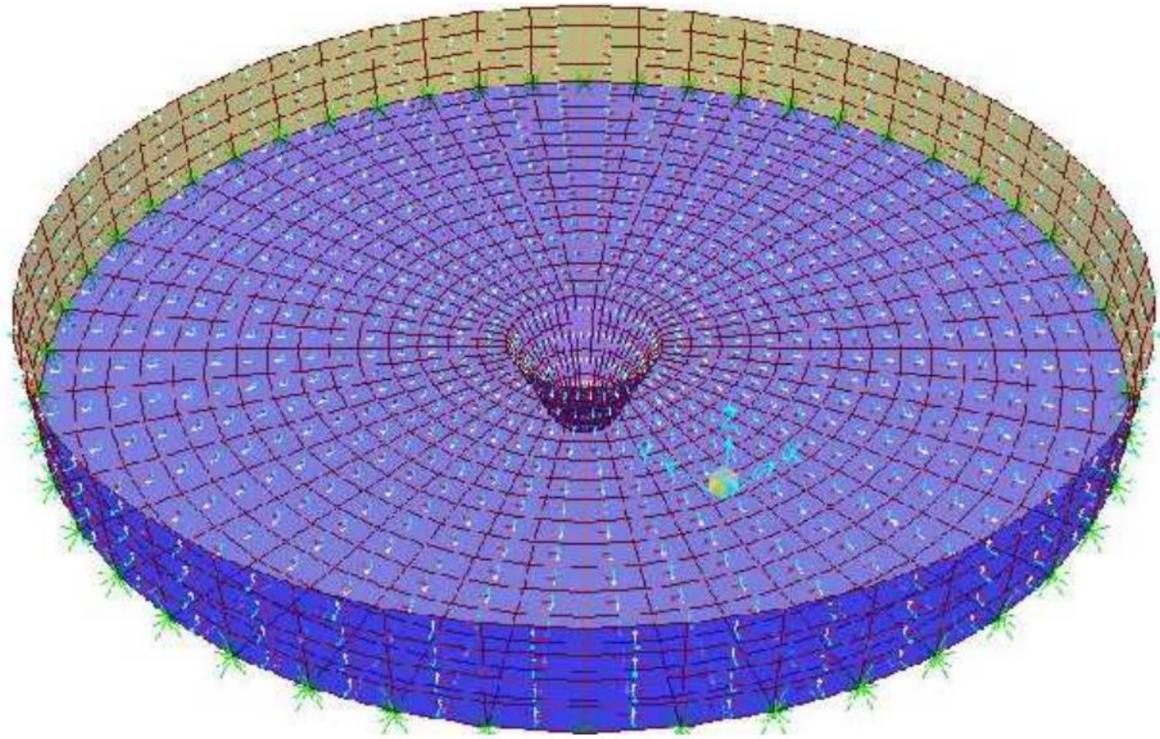


FIGURE-4:3-D Model of Secondary Clarifier

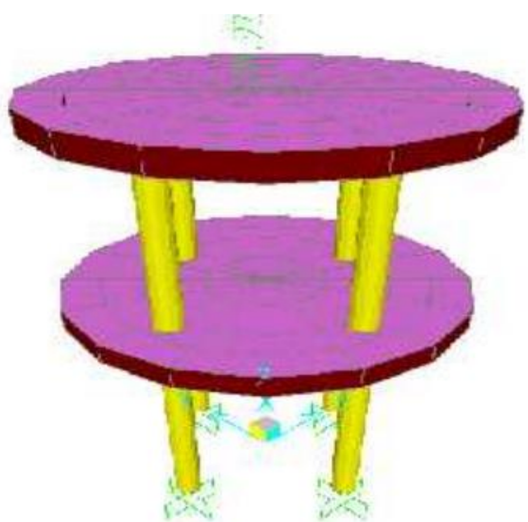


FIGURE-5:3-D Slab (S1 and S2) and Column (C1)

4. ESTIMATION OF SERVICE AND ULTIMATE LOADS

Estimation of the service and ultimate loads was concerned with calculation of the loads that are exerted directly or indirectly onto the structures. The loads that act onto the tank were estimated for purpose of stress analysis and subsequent designing of its sections.

The service loads were used for crack width analysis. First the service loads were estimated followed by calculation of the ultimate loads by multiplying the service load with their load factors.

The load factors were adopted from ACI 350M-06

The following loads have been considered:-

- Self -weight of the Secondary Clarifier(D),
- Hydrostatic load(F),
- Equipment Loads(Ls),
- Load due temperature change(T),
- Seismic inertial load(Di),and
- Hydrodynamic Load (Fi).

The loads combine in following way to give U as based on recommendation given by the code:-

a) Service loads combination

- D+LS ,
- D,and
- F+D.

b) Ultimate loads combination

- 1.2D+1.6Ls,
- 1.4D,and

- 1.4F+1.4D
- 1.2F+1.2D+Fi+Di.
- Seismic inertial load(Di),and
- Hydrodynamic Load (Fi).

As per ACI 350M-06,required strength U for other than compression controlled sections, as defined in below, shall be multiplied by the following environmental durability factor (Sd) in portions of an environmental engineering concrete structure where durability, liquid-tightness, or similar serviceability are considerations. In the case of shear design, this factor is applied to the excess shear strength carried by shear reinforcement only. This durability factor has not be used for designs using service loads and permissible service load stresses. For applicable use of the environmental durability factor (Sd) in conjunction with load combinations that include earthquake loads.

Different values of Sd has been used for different load combinations and based on spacing and bar diameter proposed for critical sections in question.

As per ACI 350M-06(9.2.6)

Where

fs max = maximum allowable stress in reinforcement at service load, Mpa.

s = center-to-center spacing of deformed bars(mm),

db =bar diameter(mm)

β = ratio of distances to the neutral axis from the extreme tension fiber and from the centroid of

the main reinforcement and it is 1.35.

h =thickness of concrete section,

Φ =factor taken to be 0.9,and

γ=load factor depends on load combinations but take generally taken to be 1.4.

The different sd used determination of ultimate loads are presented in Table-1 here below

Table-1: Environmental Durability Factor

S	d_b	$f_{s\ max}$	S_d
100	10	279.03	1.15
	12	276.27	1.16
	14	273.54	1.17
	16	270.85	1.18
	20	265.56	1.21
150	10	223.0	1.44
	12	221.59	1.45
	14	220.17	1.46
	16	218.76	1.47
	20	215.94	1.48
200	10	181.73	1.77
	12	180.96	1.78
	14	180.19	1.78
	16	179.41	1.79
	20	177.85	1.8

4.1 Hydrostatic Load

The hydrostatic loads were calculated as linearly distributed loads on respective walls and the unit weight of the waste water has been taken to be 10KN/m^3.For estimation of hydrostatic force, full water level which is at equal level to top elevation of tank has been taken. The hydrostatic load has been directly applied onto the 3-D finite element model.

4.2 Earth Pressure Load

No significant earth pressure is expected to acts on walls of Secondary Clarifier.

4.3 Loads due to Temperature Change and Shrinkage

Deformation Loads are expected to be imposed onto the secondary clarifier due to seasonalor/and daily temperature change from 100 to 250. It has been assured that minimum Re-bars recommended by ACI required to resist the loads is provided.

Accordingly, minimum horizontal re-bar ratio of 0.005 has been provided for peripheral walls the walls .Minimum vertical re-bar of 0.003 has also been provided for as vertical bars for the peripheral walls. Table-2 here below shows minimum bars to be provided at various location of Secondary clarifies.

Location of sections		Thickness of the sections(mm)	Minimum Re-bar ratio	Amount of re-bar Required(mm2/m) (each face of wall or top and bottom slabs)	Amount of re-bar Provided(m m2/m)	Re-bar Dia. And C/C selected	Distribution of Re-bars
Peripheral wall	Vertical wall	300	0.005	750	753.6	Φ12C/C150	Horizontal bar along both faces
			0.003	450	523.33	Φ10C/C150	Vertical bars along both faces
Foundation slab	Horizontal slab Under walls	300	0.005	750	753.6	Φ12C/C150	Circular bars Along top and bottom
			0.004	600	638.47	Φ10C/C300 + Φ12C/C300	Radial bars Along top and bottom faces
	Nearly Horizontal slab	250	0.005	625	753.6	Φ12C/C150	Circular bars Along top and bottom faces
			0.004	500	523.33	Φ10C/C150	Radial bars Along top and bottom faces
Hopper	Conical wall	300	0.003	450	565.2	Φ12C/C200	Nearly Vertical bars along both faces
		300	0.005	750	753.6	Φ12C/C150	Circular bars
	Circular bottom slab	300	0.003	450	565.2	Φ12C/C200	Horizontal bar along both top and bottom faces
U-channel	slab	200	0.003	300	392.5	Φ10C/C200	Horizontal Radial bar along both top and bottom faces

		200	0.005	500	523.33	Φ10C/C150	Horizontal curved bar along both top and bottom faces
	Vertical walls	200	0.003	300	392.5	Φ10C/C200	Vertical bars along both faces
		200	0.005	500	523.33	Φ10C/C150	Horizontal curved bar along both top and bottom faces
Circular slabs S1 and S2		300	0.003	450	523.33	Φ10C/C150	Horizontal bar along both top and bottom faces
Columns C1	Support for Circular slab S1 and S2	300	0.01	706.5	791.28	7Φ12	Vertical bars

Table-2: Minimum re-bars to be provided for walls and slabs of Secondary Clarifier

4.4 Seismic Inertia Load

The inertia seismic load the Secondary Clarifier has been calculated using the software

SAP2000.For estimation of the seismic inertial loads, Modal response spectrum method has been used. The peak ground acceleration in two orthogonal horizontal directions and one in vertical direction have been used to define, design response spectrum for the Secondary clarifier.

The response spectrums in the corresponding directions were determined based on the recommendations of EUROCDE-8 for soil foundation and 5% damping ratio.

As per the code, for purpose of seismic inertia analysis, the magnitude of the vertical component will be 0.9 times of the vertical component. So, the magnitude of vertical ground acceleration was specified to have peak ground acceleration magnitude of 0.045g for peak ground acceleration of value 0.05g.

Using the peak ground accelerations, design response spectra or target response of the earth quake were determined. The design response spectra has been derived by multiplying the peak ground acceleration with standard shape of response spectra which are available in Eurocode-8.

5. Responses of the Structures to the Loads

The loads determined in previous sections were applied to the 3-D finite element model of the structure. The responses of the structure were estimated with help SAP2000 analysis software. The Table-6 and Table-7 shown here below are used to summarize the response of the structure in form of maximum bending moment (M11 and M22), maximum shear force (V23 and V13). M22 and M11 are moments for which re-bars are provided in direction of local axis-1 and local axis -2.

Local axis-1 and Local axis-2 are shown by red and white arrows respectively, see figure-4. V23and V13 are shear forces corresponding to M22 and M11 .T is hoop stress acting tangentially at center of wall. Compress forces acting at the center of walls and columns are reported as -T if there is any.

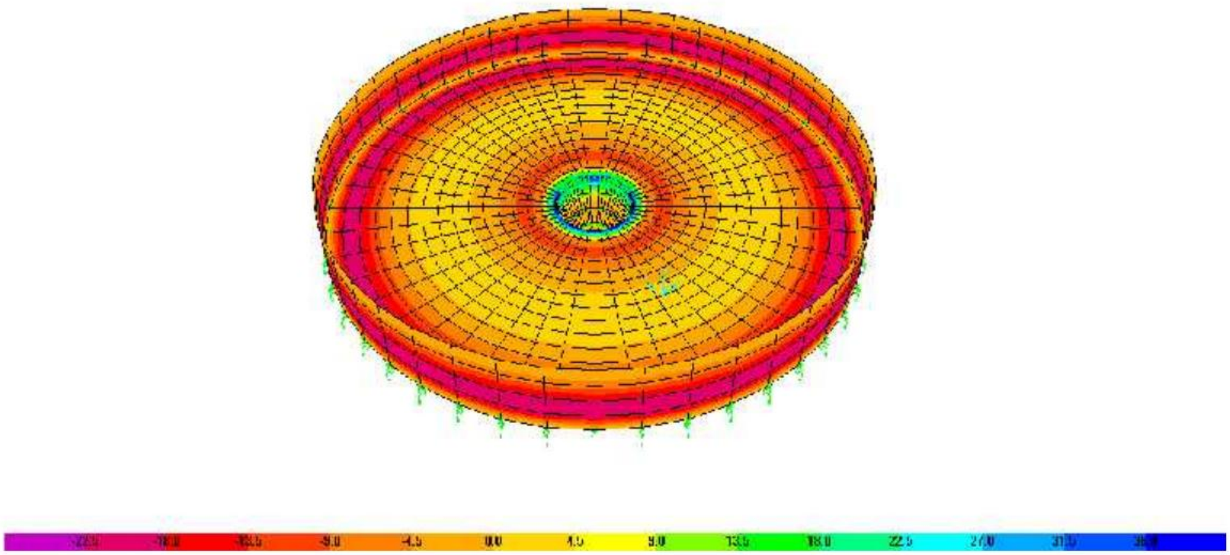


Figure-8: Wall and foundation slab of Secondary Clarifier: Bending Moment (M_{22} KN.m/m) due to 1.4(D+F)

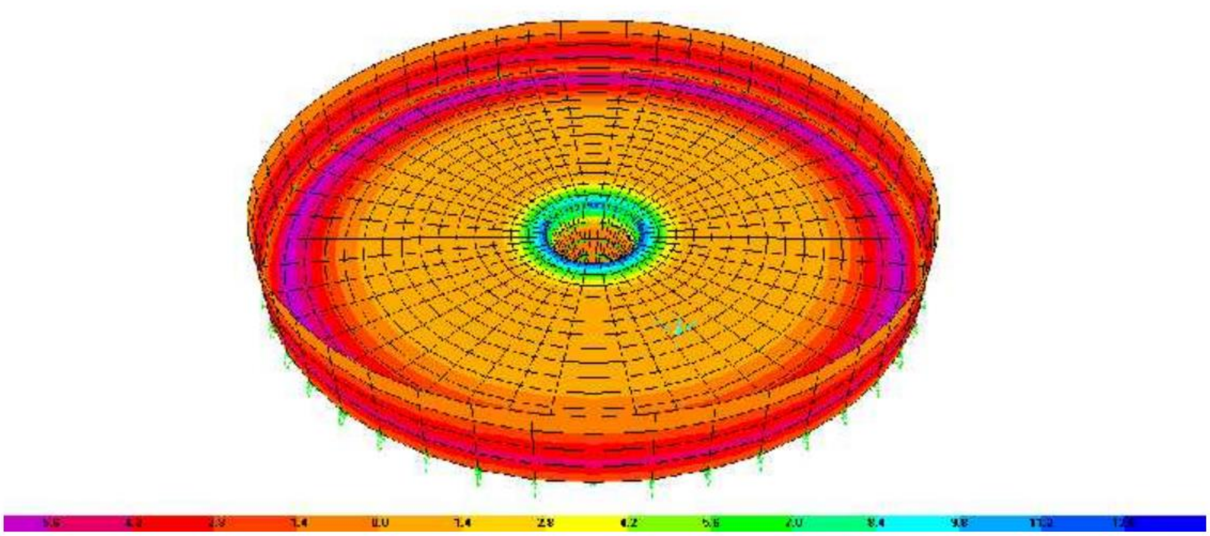


Figure-9: Wall and foundation slab of Secondary Clarifier: Bending Moment (M_{11}KN.m/m) due to 1.4(D+F)

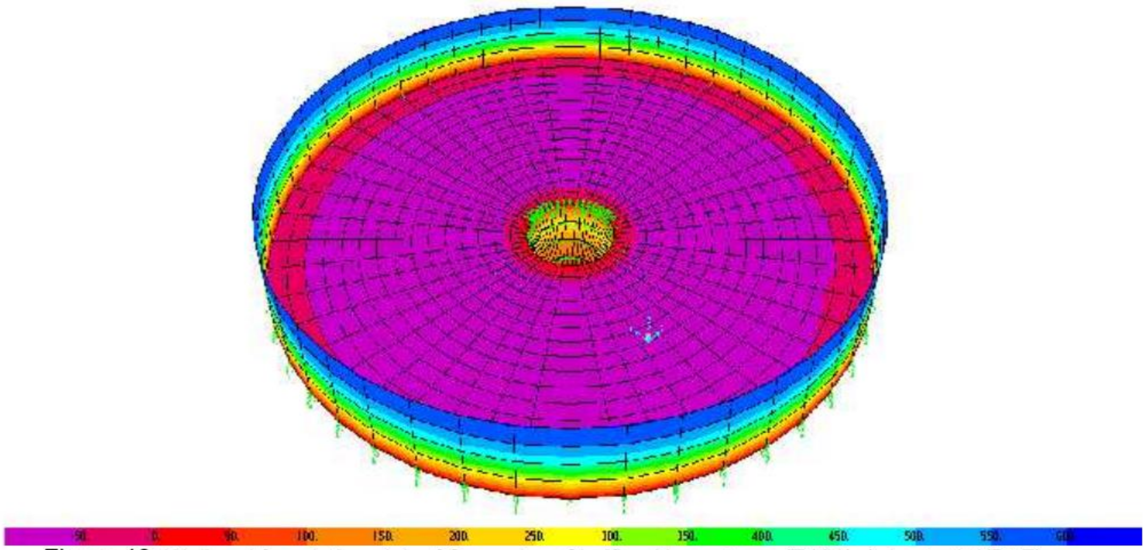


Figure-10: Wall and foundation slab of Secondary Clarifier: Hoop stress (T KN/m) due to 1.4(D+F)

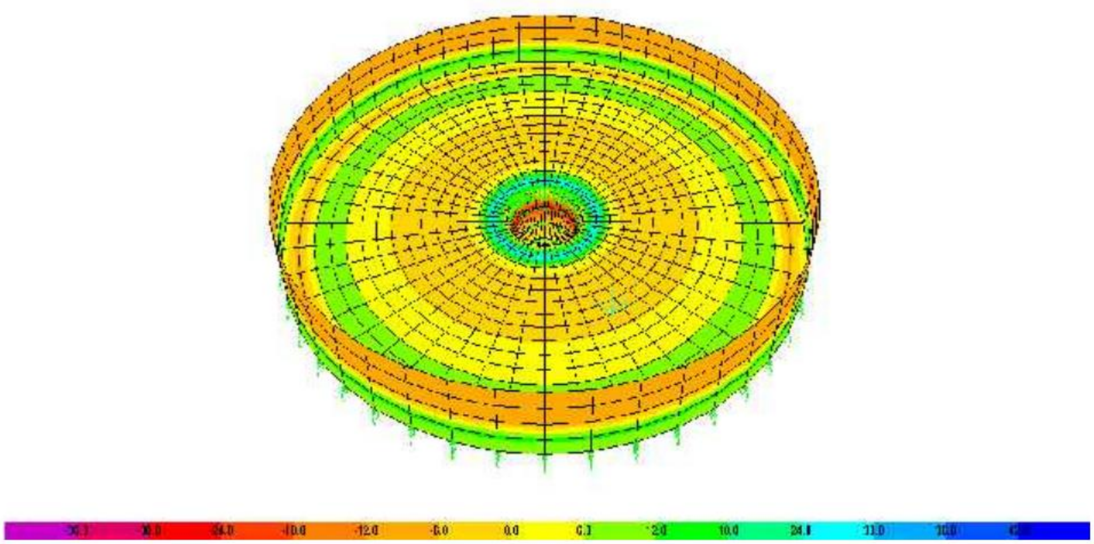


Figure-11: Wall and foundation slab of Secondary Clarifier: Shear force (V_{23} KN/m) due to 1.4(D+F)

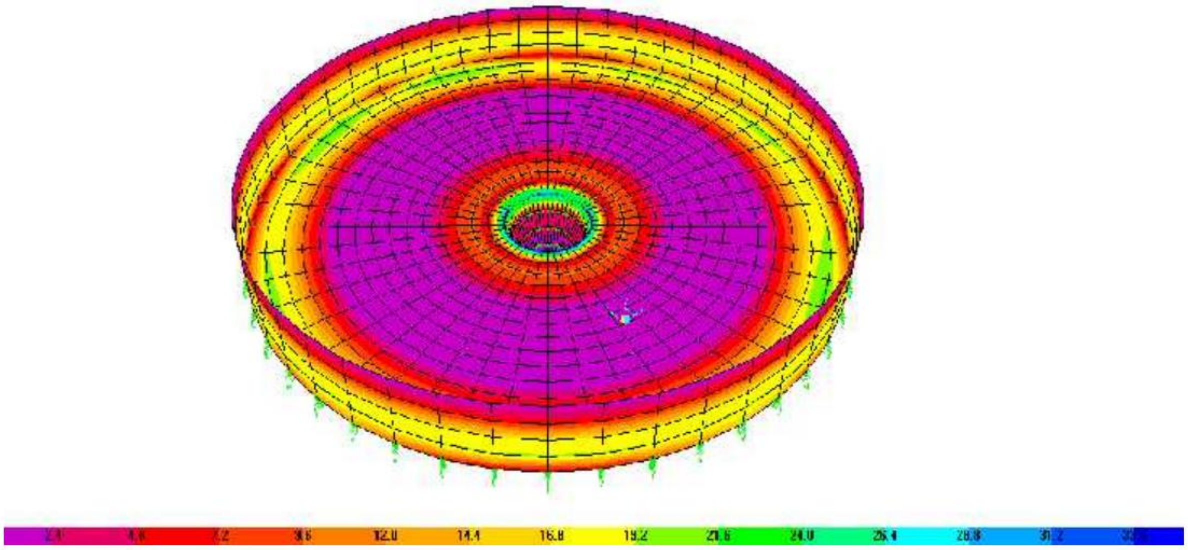


Figure-12: Wall and foundation slab of Secondary Clarifier: Bending Moment (M_{22} KN.m/m) due to 1.2(D+F)+F_i+D_i

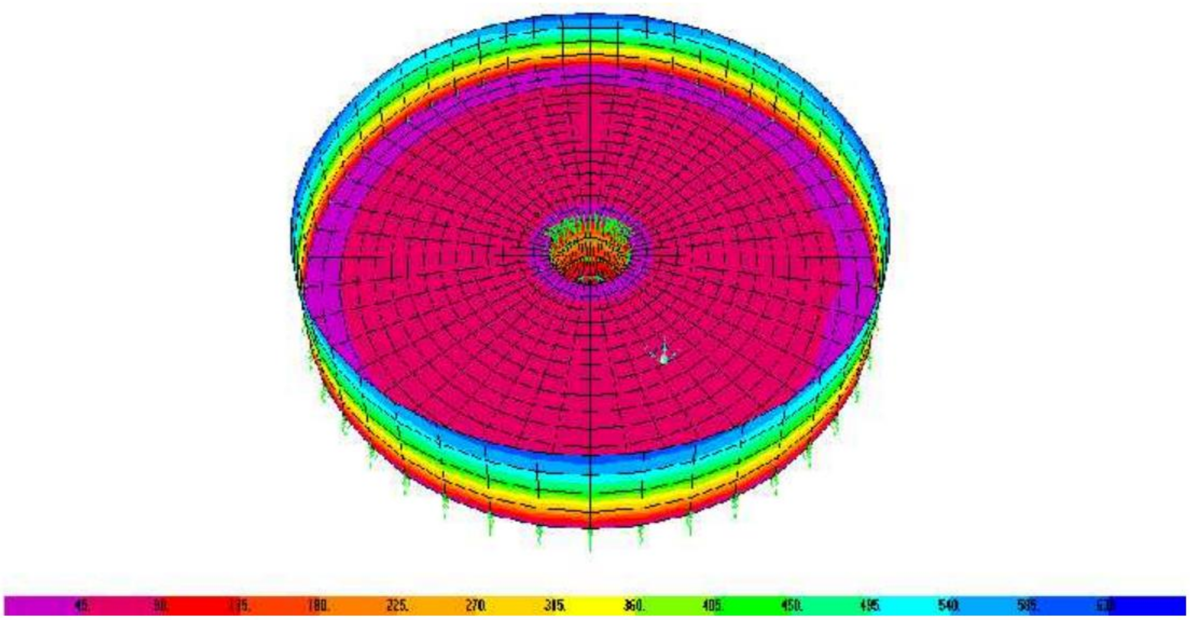


Figure-13: Wall and foundation slab of Secondary Clarifier: Hoop Stress (T KN/m) due to 1.2(D+F)+F_i+D_i

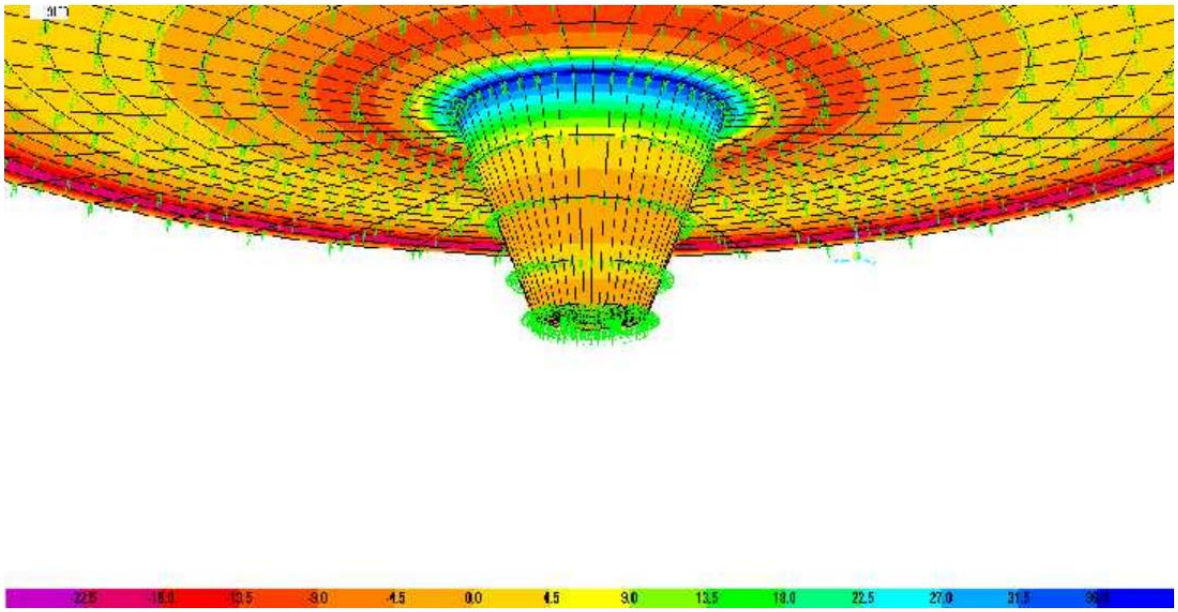


Figure-14: Hooper wall of Secondary Clarifier: Bending Moment (M_{22} KN.m/m) due to 1.4(D+F)z

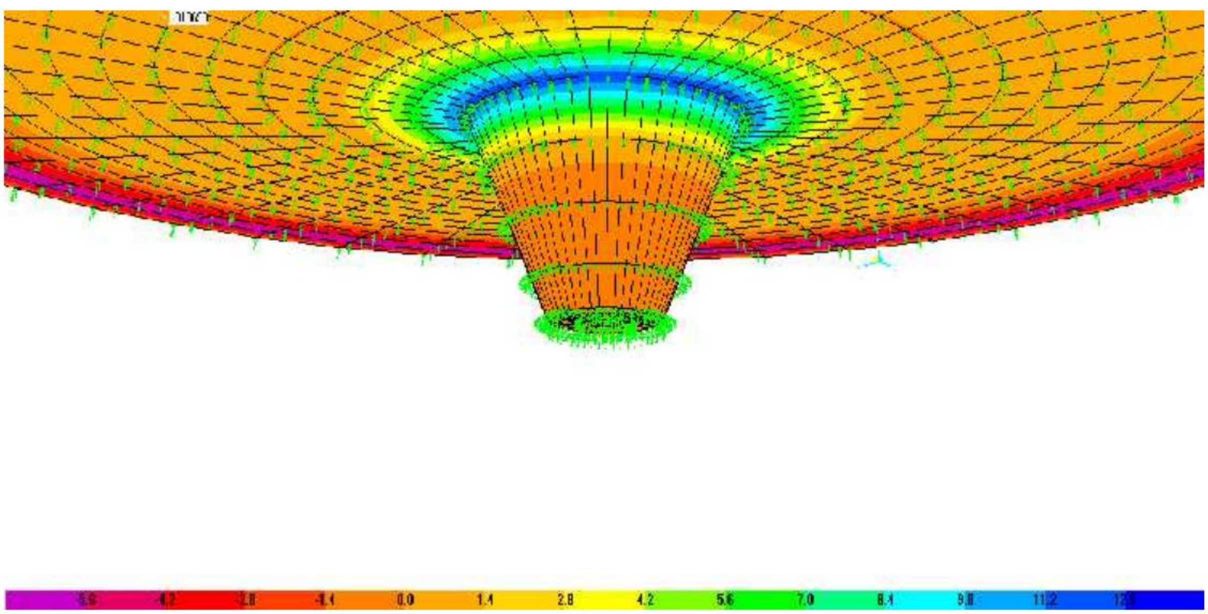


Figure 15: Hooper wall of Secondary Clarifier: Bending Moment (M_{11} KN.m/m) due to 1.4(D+F)

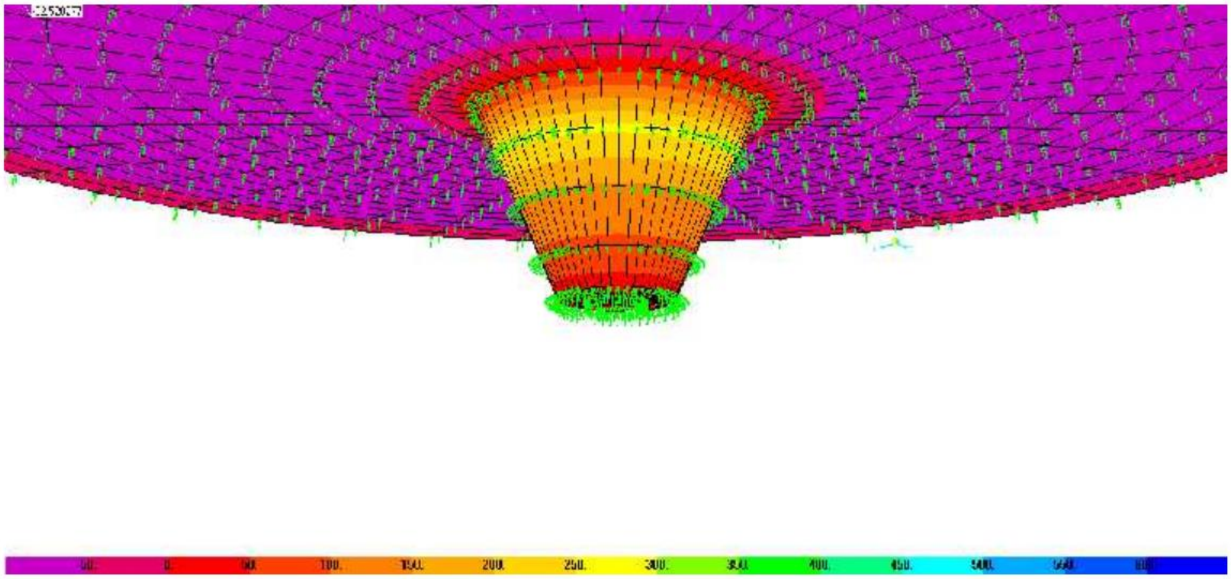


Figure-16: Hooper wall of Secondary Clarifier: Hoop stress (T KN/m) due to 1.4(D+F)

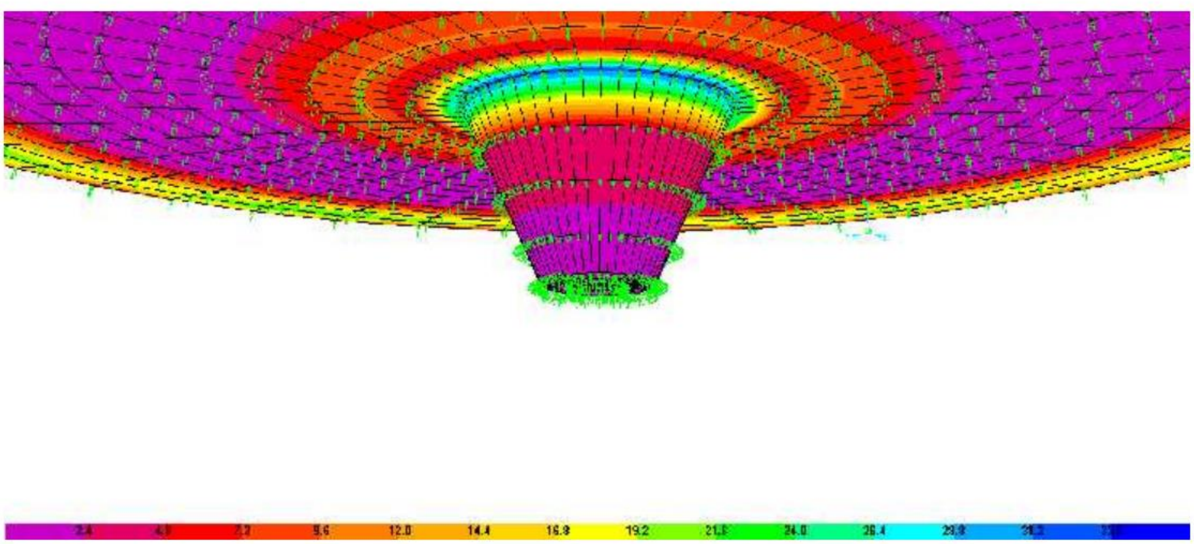


Figure-17: Hooper wall of Secondary Clarifier: Bending Moment (M_{22} KN.m/m) due to 1.2(D+F) + F_i+D_i

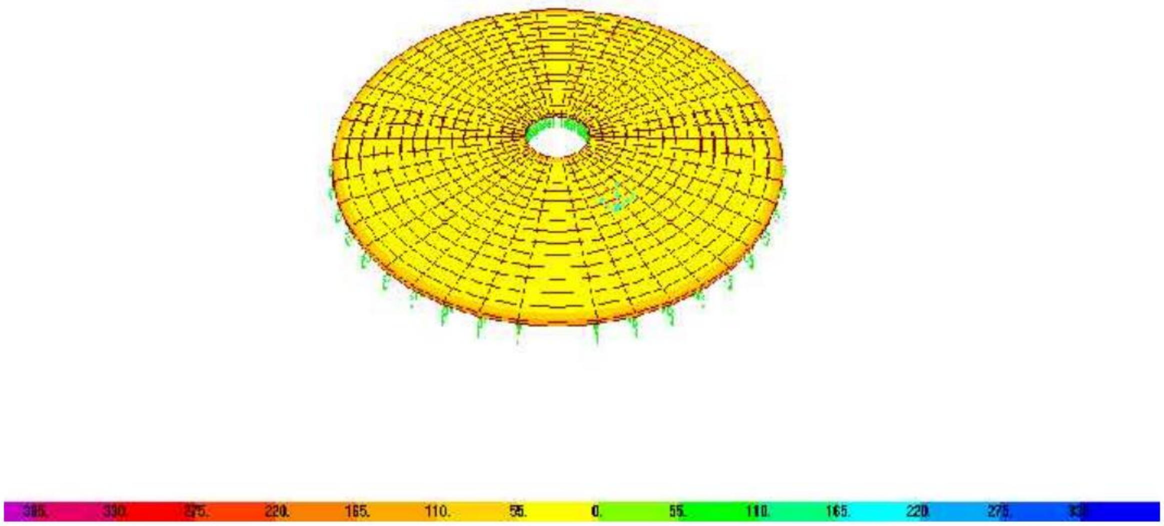


Figure-20: Secondary Clarifier foundation Slab : Bearing Pressure (KN/m2) due to (D+F)

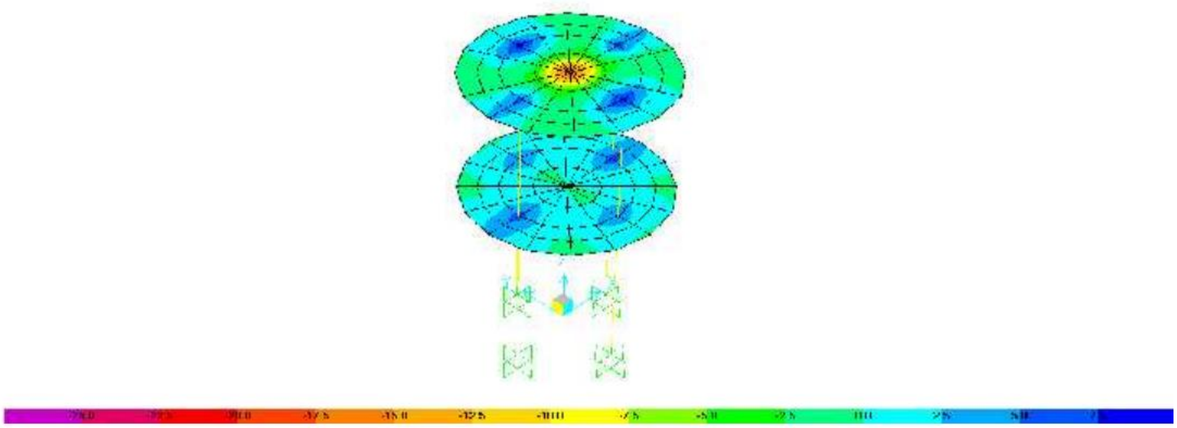


Figure-21: Circular slab S1 and S2 : Bending Moment (KN.m/m) due to (1.2D+1.6Ls)

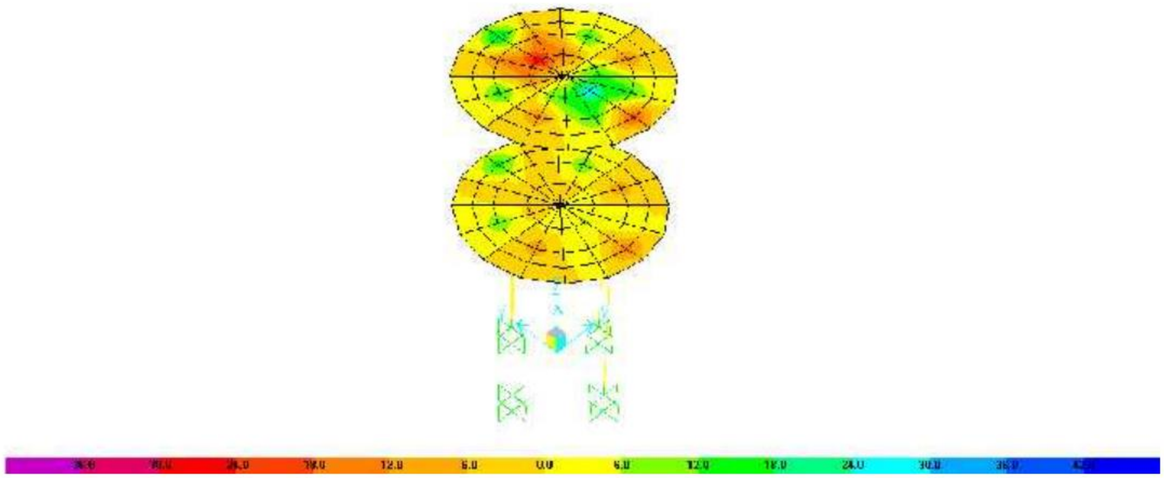


Figure-22: Circular slab S1 and S2 : ShearforceV23 (KN./m) due to (1.2D+1.6Ls)

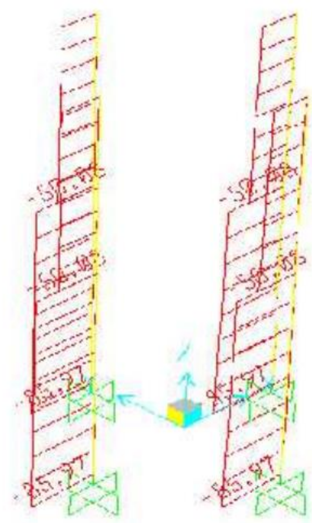


Figure-23: Columns C1: Axial force-T (KN./m) due to (1.2D+1.6Ls)

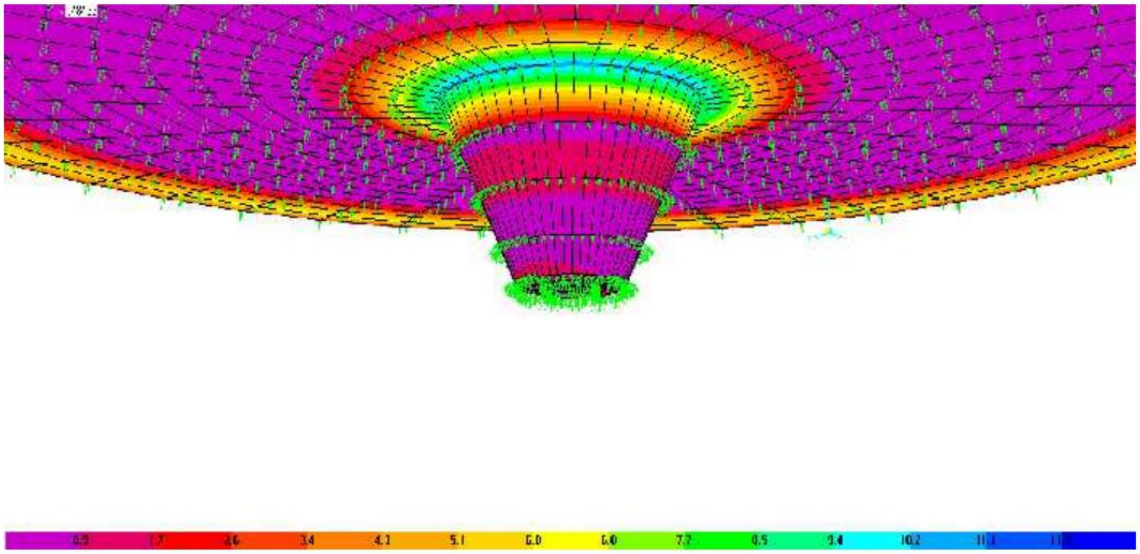


Figure-18 Hooper wall of Secondary Clarifier Bending Moment (M_{11} KN.m/m) due to 1.2(D+F) + F_i+D_i

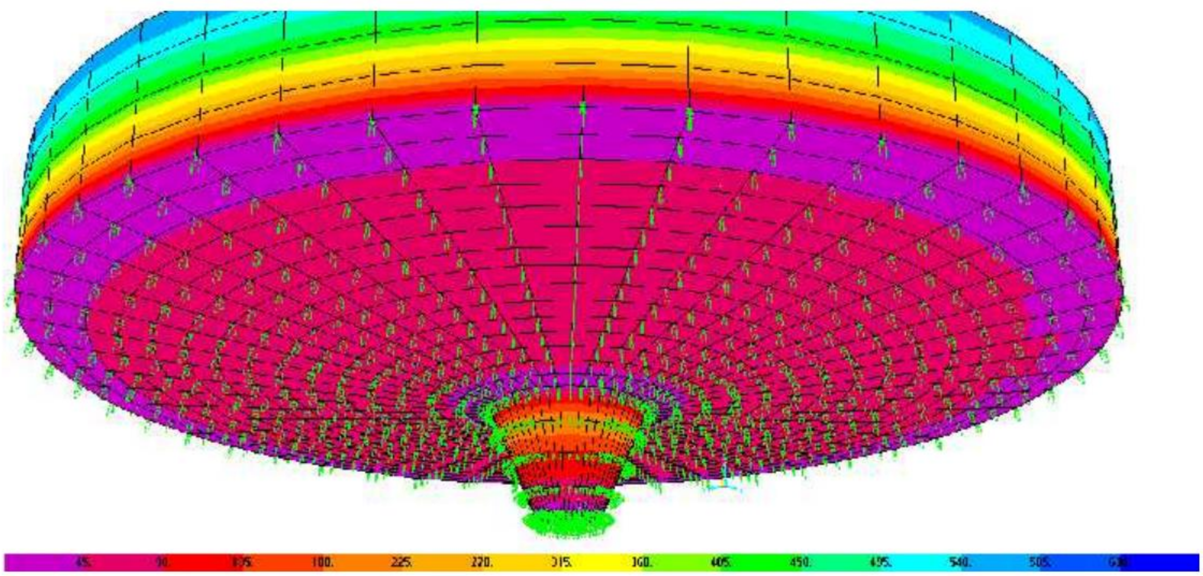


Figure-19: Hooper wall of Secondary Clarifier: Hoop Stress (T KN/m) due to 1.2(D+F) + F_i+D_i

Critical Locations	Load Combinations	Bending moment (KN-m/m)			Shear forces KN/m		Axial force (KN/m)	see the figure
		M_{22}	M_{11}	M_{33}	V_{23}	V_{13}	T	
Peripheral Walls	1.4F+1.4D	-23	-4.5		12	-	600	8-11
	1.2(F+D)+ (Di+Fi)	20	10		31	9	615	12-13
Foundation slab	1.4F+1.4D	23	-5.6	-	30	-		8-11
	1.2(F+D)+ (Di+Fi)	27	6	-	26	6.6	28	12-13
Hooper conical walls	**1.4F+1.4D**	36	13	-	30	2.26	32	14 -15
	1.2(F+D)+ **(Di+Fi)**	33.6	12		25	-	250	16-19
Hooper foundation slab	**1.4F+1.4D**	-	2.3	-	-	3.85	28	-
	1.2(F+D)+ **(Di+Fi)**	1.5	2.6		3.5	2.6	45	-
Circular slab(S1 and S2)	**1.2D+1.6Ls**	19	19		22	22		21-22
Column(C1)	**1.2D+1.6Ls**						-85.97	23

Table-7: Maximum Response of Secondary Clarifier to service load.

Critical Locations	Load Combinations	Bending moment (KN-m/m)			Shear forces KN/m		Axial force(KN/m)	see the figure
		M_{22}	M_{11}	M_{33}	V_{23}	V_{13}	T	
Peripheral walls	F+D	12	2.3				288	
Foundation slab	**F+D**	10	6.3				16	
Hooper Conical wall	**F+D**	19	6				128	

6 Designing of Structural Members

Walls and slabs of Secondary Clarifier have been designed to with stand stresses induced by both the service load and ultimate load. For ultimate load, the critical sections were designed to with standard moments and shears. Moreover, the adequacy of re-bars provided, for purpose of withstanding ultimate loads, for controlling stress induced by temperature and shrinkage has been carried out. If they are in adequate, then minimum re-bars have been provided.

For service loads, checking adequacy of the re-bar for limiting crack width to 0.2mm has been conducted.

For designing purpose, 75% of the maximum bending moments (ultimate Loads and service loads) as calculated and given in previous section was utilized to design critical sections with exception of long cantilever walls or perimeter walls and other members acting so.

6.1 Designing of Structures for Ultimate Loads

Walls, foundation slab, and columns have been identified as major structural members of the structures. The members have been designed for the ultimate bending moments, shear forces, and hoop stresses.

Member design of structural elements has been carried out for all ultimate load combination described here. The member design evolved determination of economical sections and corresponding reinforcement that enable the structural member to withstand the ultimate loads.

6.1.1 Designing of Walls ,Foundation Slab and Columns

Vertical walls and foundation slabs have been designed for flexure using 1000mm wide strip(b) and their respective thickness (h) and applying design relations set out for beam. In designing the walls for a factored negative or positive moment, **M_{22}** or **M_{11}**, the depth of the compression block are given by **a.**Table-8 here below summarizes the designing process.

$$a = d - \sqrt{d^2 - \frac{2|M_u|}{0.85 f'_c \phi b}}$$

Where d is effective depth,

Mu is ultimate moment

M22 or M11 and Φ are strength reduction factors.

The following value has been adopted for designing:-

Φ=0.9 for flexure (tensioned controlled),

Φ=0.75 for shear.

The reinforcement area (As) required to resist tensile force caused by Mu has been determine as follows:-

$$A_s = \frac{M_u}{\phi f_y \left(d - \frac{a}{2} \right)}$$

Whether As is above the Maximum allowable limit has been checked in such way that its geometric reinforcement ratio (ρ) is less than the maximum geometric ratio (ρmax =0.012).The maximum geometric ratio is taken to 75% of the balanced geometric ratio (ρb).In turn the balanced geometric ratio has been obtained using the following relation

$$\rho_b = 0.85\beta_1 \frac{f'_c}{f_y} \frac{\epsilon_u}{\epsilon_u + \epsilon_y}$$

Where,β1 is factor=0.85,

f'c is cylindrical compressive strength of age 28 days,

fyis minimum yield strength of re-bar,

εu is maximum compressive strain of concrete=0.0035,and

εy is minimum yield strength of re-bars.

Moreover, the following relation has been employed to design members for shear (ACI11.2.1.1):-

$$V_c = \lambda \frac{\sqrt{f'_c}}{6} b_w d$$

$$V_u \leq \phi V_c$$

Where

VC is Nominal shear strength of plain concrete in (N), Vu is design shear force (V23 or V13) at section of interest,

λ is 1.0 for normal concrete and 0.75 for light weight concrete,

f'c is cylindrical strength of concrete at age of 28days (Mpa.)

bw is width of concrete section(mm),and d is effective depth of concrete section (mm)

Φ is strength reduction factor, which is 0.75 for shear strength

The design for shear is summarized in Table-9 here below.

The following relations have been employed to determine re-bars in walls of the Secondary Clarifier peripheral walls for hoop stress.

$$\mathbf{A_s=T/(0.9f_y)}$$

Where

As=is amount re-bar required to take up stress,

T is factored hoop stress, T=615KN and

fyis minimum yield strength of re-bar.

As=615000N/(0.9*500N/mm2)=1366.67mm2 for both faces,683.33mm2 on each face which is less than the minimum(753.6mm2). This re-bar shall be added to bars from M11 (8.25Kn.m/m) and checked again if it is less than minimum. The total will be=683.33mm2+75.42mm2=758.75 mm2

This is almost equal to the minimum re-bar. Therefore, minimum horizontal re-bar is provided at both faces of peripheral walls.

The above relation has been developed based on that assumption that concrete is not effective to take-up the hoop stress and it is only the re-bars that resist the hoop stress.

Table-8: Flexural Design of Re-bar for Walls, and Foundation Slab

Location of critical section	Mu (KN m/m)	h (mm)	d (mm)	b (mm)	a (mm)	ρ	As Required (mm2)/m	As Provided (mm2)/m	Remark
Peripheral Walls	23	300	245	1000	5.16	0.00086	210.84	523.33 (Φ10c/c150)	Vertical bars
Foundation slab	23	250	195	1000	6.53	0.00137	266.57	638.47 (Φ12c/c300+ Φ10c/c300)	Radial Bars
Hooper conical walls	36	300	245	1000	8.138	0.00135	332.045	565.2 (Φ12c/c200)	Inclined bars

Moreover, the relation here below has been employed to design the columns (C1) that supports slabs S1 and S2.

Pu≤ ΦPn (max) where

P

u=Design factor axial load (N) , Pn(max) nominal axial strength of section(N), and Φ strength reduction factor=0.65.

ΦPn(max)=0.80Φ(0.85f’c(Ag-Ast)+fyAst).

Where

Ag=gross area of section (mm2), and

Ast=area of Re-bar (mm2).

For column dia. of 300mm, Ag=706550mm2, for 7Φ12, as=791.28mm2, fy = 500N/mm2,

f'c=24N/mm2.

ΦPn (max) =0.8*0.65*(0.85*24*(70650-791.28) +500*791.28) =946794.102N (946.79KN).

However, Pu=85.59KN (Table-6) which is well less than ΦPn(max).

Therefore, the re-bar provided to column C1 and dimension of the same are enough to support slabs loads coming from the scrapper.

Table-9: Checking Shear Strength of the Critical Section against V23

Location of critical section	**Vu** (**V_{23}**) (KN/m)	**h** (mm)	**d** (mm)	**b** (mm)	**Φ V_C** (KN/m)	Remark
Peripheral Walls	31	300	245	1000	127	The section is thick enough to carry the design shear force
Foundation slab	30	250	195	1000	118.7	It is ok
Hooper conical walls	30	300	245	1000	127	It is ok

The wall of Secondary Clarifier is subjected to ring or hoop tension due to the internal restraint of re-bars to concrete shrinkage and hoop stress due to external load. The hoop stress due to concrete shrinkage and hoop stress due to service load should not cause excessive stress in the concrete. Therefore, it should be made sure the total hoop stress does not cause excessive tensile stress on concrete during shrinkage. The hoop tension stress on concrete must not be greater than 2400KN/m^2.

The relation given here below is used to calculate the total hoop stress.

$\mathbf{f_{cs}}$= [$\mathbf{CE_sA_s}$ + $\mathbf{T}$]/[$\mathbf{A_c+nA_s}$]

where

$\mathbf{f_{cs}}$ is Tensile stress due to shrinkage,

T=hoop stress.

C=coefficient of Shrinkage (0.0003),

$\mathbf{A_c}$=concrete cross section,

$\mathbf{A_s}$=area re-bar,

n=modular ratio of concrete=6.9, and

$\mathbf{A_s}$=753.6mm2,

$\mathbf{A_c}$=300000mm2.

The peripheral wall of Secondary Clarifier is exposed to service load that causes hoop stress. Therefore, **T** is equal to 288*0.75=216KN. As result, the tensile stress acting on the concrete is 2189.25KN/m2 which is less than the allowable tensile stress (2400KN/m2). The maximum bearing pressure at the base of the foundation slab below the peripheral wall is found to be about 130K/m2, see figure-20. The induced foundation pressure is well below the allowable pressure of the back filled foundation material (285Kpa.)